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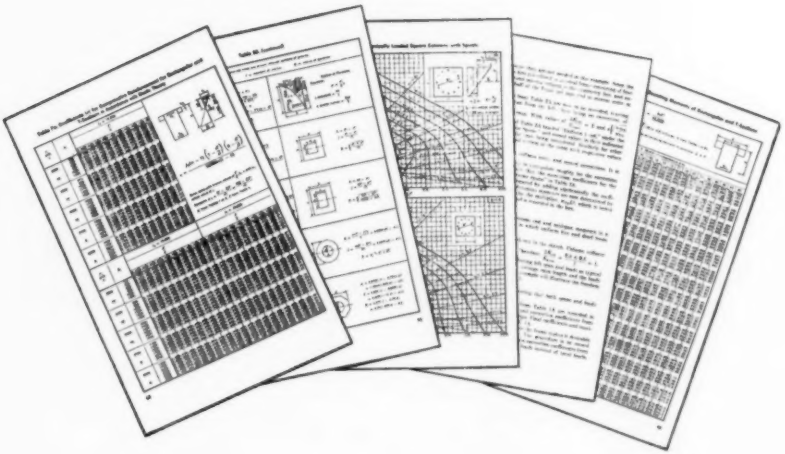

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Coming next month in the JOURNAL

Report of Committee Joint ACI-ASCE 323 entitled "TENTATIVE RECOMMENDATIONS FOR PRESTRESSED CONCRETE" will appear in the January issue. An outgrowth of the unpublished progress report presented by the committee at the 1957 convention in Dallas, it has been approved for publication as a committee report only.

•

"MULTISTORY LIFT-SLAB CONSTRUCTION" by W. Sefton discusses the general method, its advantages and disadvantages; outlines lifting sequence of a 12-story unit constructed in Toronto; describes slabs, lifting collars, and columns.

•

The results of a series of experiments on "RELATION OF ELEVATED TEMPERATURES OF PORTLAND CEMENT MIXTURES TO SURFACE REMOVAL" have been compiled by R. H. Heiskell, R. H. Black, R. J. Crew, and H. Lee.

They compare effect of oven heating with several surface flame treatments in producing a surface layer of concrete that can be easily removed by abrasion.

•

"PROPORTIONING AND CONTROL OF LIGHTWEIGHT STRUCTURAL CONCRETE," by George H. Nelson and Otto C. Frei, sets forth the experiences of the authors with lightweight aggregate concrete.

Method of mix proportioning is detailed, along with recommendations for field control. Tests on compressive, flexural, bond, and diagonal tension strength, modulus of elasticity, and thermal conductivity of the lightweight concrete are reported. Comparison tests were made of mixes with cement factor from 3 to 9 bags per cu yd.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

Secretary, Technical Activities Committee
AMERICAN CONCRETE INSTITUTE
P. O. BOX 4754, REDFORD STATION, DETROIT 19, MICHIGAN

Proposed Revision of ACI Standard 711-53

Minimum Standard Requirements for Precast Concrete Floor and Roof Units*

Reported by ACI Committee 711

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SYNOPSIS

Minimum standard requirements for single units or multiple element assemblies, to be used in conjunction with ACI 318-56. Covers materials; design principles; manufacture including curing and handling; testing of completed units; installation plans; and special provisions for holes and openings in members. Design chapter treats such problems as dimensions, allowable deflection, structural concrete topping, reinforcement anchorage and location, and use of lightweight concrete.

CHAPTER 1—GENERAL

101—Scope or limits

(*a*) These minimum standard requirements for precast concrete floor and roof units are to be used in conjunction with the "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

(*b*) Precast floor and roof units which are manufactured as single units, or multiple element assemblies, are to be designed in accordance with standard reinforced concrete theory and as set forth in ACI 318-56 except that the requirements for concrete protection for reinforcement, bar spacing, bond stress, and anchorage may be modified as in Sections 402(a), 405(a), and 406(a) for units made by factory methods and under factory control.

(*c*) With reference to precast systems the design of which does not conform to Section 101(b), the sponsors have the right to present for acceptance the data upon which their design is based, in accordance with the provisions of Section 103(a) of ACI 318-56.†

*Title No. 54-24 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, Dec. 1957, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

The report was unanimously approved by the committee listed above. It is released by the Standards Committee for publication and discussion with a view to its consideration for adoption as an Institute Standard to supersede "Minimum Standard Requirements For Precast Concrete Floor Units (ACI 711-53)" at the 54th Annual Convention, Chicago, Ill., Feb. 24-27, 1958.

†See appendix hereto, p. 448.

CHAPTER 2—MATERIALS

201—Aggregate

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (ASTM C 33-56T) and "Tentative Specifications for Lightweight Aggregate for Structural Concrete" (ASTM C330-53T) provided, however, that aggregates which have been shown by test or actual service to produce concrete of the required strength, durability, watertightness, fire resistance, and wearing qualities may be used under Section 302(a) Method 2, ACI 318-56, where authorized by the building official.

(*b*) The maximum size of the aggregate shall in no case be larger than three-fourths of the minimum clear distance between reinforcing bars and sides of the forms, nor larger than one-third of the narrowest dimension between sides of the forms in which the unit is cast, except that where concrete is placed by means of high-frequency vibration the maximum size of the aggregate shall not be larger than one-half the narrowest dimension between the sides of the forms.

202—Concrete

(*a*) The quality and strength of concrete for floor or roof units made of sand and gravel, crushed stone, air-cooled slag, or other heavy aggregate shall be as provided in Sections 301 and 302 of ACI 318-56.

(*b*) Lightweight concrete for roof or floor units, which is made either by the introduction of air bubbles into the mix (cellular concrete) or by the use of lightweight aggregates, is permitted if the allowable unit stresses used in design satisfy the requirements of Section 401.

203—Reinforcement

(*a*) In the non-prestressed types of floor or roof units the reinforcement shall conform to the specifications outlined in ACI 318-56, Section 208. The allowable stresses shall conform to the requirements set forth in Section 306(a) and (b) of ACI 318-56.

(*b*) Fully* prestressed units shall employ high tensile strength strand or wire, bonded to the concrete over the full length of the unit.†

CHAPTER 3—TESTS

301—Tests of an individual flexural unit

(*a*) When an individual floor or roof unit is to be tested as a simple span beam, the zero for deflection shall be under the design total dead load to be carried. The maximum 24-hr midspan deflection due to a test load of twice the design live load‡ shall not exceed 1/160 of the span, and the immediate recovery upon removal thereof shall be at least 60 percent. Such units shall

*For the use of mild steel bars for partial prestressing of units see Section 406(a).
†Refer to the Dec. 15, 1956 draft report of ACI-ASCE Joint Committee 323, "Recommended Practice for Prestressed Concrete," unpublished.
‡With minimum test loads of 80 and 60 psf for floors and roofs respectively.

then be tested to complete failure,* which shall not occur at less than two times the sum of design dead and live load, nor less than three times the design live load alone.

302—Manufacturer's certification

(*a*) A manufacturer whose standard precast floor and roof units have been designed and manufactured in conformity with this ACI standard, and who so represents, shall make available to the prospective customer (or his technical representative) a report which so indicates. He shall also submit a copy of a test report by an approved independent laboratory or consulting engineer covering a Section 301(a) test of a unit from the same manufacturing series as the one under consideration.

303—Load tests of an existing structure

(*a*) When a field test† is to be made, the floor or roof panel shall be tested in accordance with and shall conform to the requirements of ACI 318-56 Sections 202 and 203.

CHAPTER 4—DESIGN

400—Notation

n = ratio of modulus of elasticity of steel to that of concrete
k = ratio of depth of beam neutral axis to the beam depth to centroid of the tensile reinforcement
j = ratio of distance between centroid of compression and centroid of tension to the beam depth d

401—Allowable design stresses in concrete and reinforcement

(*a*) For all concretes the design allowable stresses in the concrete and reinforcement under safe working load shall conform to the requirements set forth in Sections 305(a) and 306(a) and (b) of ACI 318-56.

(*b*) For lightweight aggregate concretes and cellular concrete the n values shall be determined as in Section 407(a), and the corresponding values of k and j computed.

(*c*) Fully prestressed units shall be designed according to well recognized prestressed beam principles and methods.‡

402—Concrete protection for reinforcement

(*a*) Precast floor and roof units made of high quality factory-controlled concrete may, when used in locations protected from the weather or moisture, be approved with $\frac{5}{8}$-in. concrete cover for the reinforcement, provided however, that the concrete cover in all cases shall be at least equal to the diameter of the bars. To insure exact final location of the reinforcement, positive and

*If no failure fracture occurs, the load causing a deflection of 1/60 of the span is to be considered the failure load.
†Herein a *field test* is a test made upon a floor or roof area in place in a structure.
‡Refer to the Dec. 15, 1956 draft report of ACI-ASCE Joint Committee 323, "Recommended Practice for Prestressed Concrete," unpublished.

rigid devices for that purpose shall be used in the manufacturing process. When the precast members are exposed to weather, moisure, or direct fire hazard the protective cover shall be increased to conform with ACI 318-56, Section 507.

403—Structural concrete toppings

(*a*) When field-placed toppings are to be relied upon in computing the strength of a floor section, they must be thoroughly bonded to the surface* of the precast unit. The topping concrete shall develop at least three-fourths of the design strength of that of the precast unit, and have its design governed by the corresponding lower allowable stresses.

(*b*) The maximum aggregate size shall not exceed one-third the thickness of the topping.

(*c*) In no case shall the calculated horizontal shearing stress within the plane of the construction joint exceed $0.02\ f_c'$ of the weaker concrete, due to the design load above it, without the use of shear reinforcement across said plane.

404—Deflections and span-depth ratios

(*a*) The permissible simple span deflections of floor and roof units are set by the test requirements of Section 301(a), but in no case shall the span-depth ratio for floor units exceed 40. For this purpose the thickness of a topping may be included in computing the depth if integral action with the precast portion has previously been achieved through control of strength and placing in conformity with the requirements of Section 403.

(*b*) Floor and roof designs involving units containing no prestress whatever shall be accompanied by the certifications of Section 302(a).

405—Flexure bar spacings and bond stress

(*a*) When the reinforcement for precast joists consists of an upper and a lower chord with properly designed web reinforcement welded thereto, and with a vertical bar of the same size as the web reinforcement (but never of less than ⅜ in. diameter) weld-connecting the chords at their ends, the 6-in. anchorage requirement of ACI 318-56, Section 902(c), and the spacing requirement of Section 505(a) may be waived, provided that the members are made under factory controlled conditions in which the concrete is vibrated or forced into place under pressure, and provided that the clear bar spacing is not less than one-half the maximum size of the coarse aggregate, and that the bond stress allowed by ACI 318-56, Section 305,† is not exceeded when computed by the formula:

$$u = \frac{V}{\frac{3}{4} \sum o j d}$$

*Satisfactory bonding requires a clean rough surface treated with neat cement grout. The topping must next be immediately placed, thoroughly compacted, and cured.

†See Appendix, p. 448.

406—Anchorage of reinforcement in partially prestressed units

(*a*) The adequacy of the anchorage of the flexure reinforcement of precast floor and roof units having no web end vertical or top reinforcement but which meet the other requirements of Section 405(a) and have flexure reinforcement tensioned in the manufacturing process (or the concrete compressed), shall be demonstrated by suitable flexure tests of the units resulting in failures in flexure or shear rather than of the anchorage.

407—Lightweight concrete *n* values

(*a*) For lightweight aggregate concrete, and for cellular concrete, the value of n to be used in designing shall be determined from rapidly* conducted compression tests of standard 6 x 12-in. cylindrical specimens from the proposed mix. In no case shall the n values used be less than $1\frac{1}{2}$ times the value specified in Section 305 of ACI 318-56 for natural stone concrete of the same compressive strength.

408—Precast joist floor design

(*a*) Precast joists that will be keyed $\frac{5}{8}$ in. into a field-placed floor slab may be erected without falsework or jacking and designed for T-beam action with the slab, provided that in designing the whole dead load is assigned to the precast joist, with only the live load considered to be carried by the combined section of increased depth.

(*b*) When the unit horizontal shear stress v_h between floor slab and joist (the joists being embedded $\frac{5}{8}$ in.) exceeds $0.03\ f_c'$ of the weaker concrete (maximum value 90 psi) due to the superimposed live load, web reinforcement must be used between such elements. See the table of ACI 318-56, Section 305(a).

(*c*) Where the ends of joists do not bear upon walls or girders, as at stair wells, steel joist hangers shall be used to provide end support, or hanger-type bent bars shall be cast into the end of the joist to be supported.

(*d*) The maximum clear distance between the permanent lateral supports of the flanges of a precast joist shall not exceed 32 times the width thereof.

409—Minimum thickness of precast joist floor slabs

(*a*) The minimum thickness of *cast-in-place* floor slabs for joist heads embedded not less than $\frac{5}{8}$ in. and for joist spacings less than 30 in., is 2 in. For joist spacing of 30 to 36 in. the minimum thickness of slab concrete floors is $2\frac{1}{2}$ in. Greater thickness may be required where unusual loads or spans are encountered. The required thickness of slabs spanning more than 36 in. shall be determined by accepted design methods, but shall not be less than $2\frac{1}{2}$ in. In the case of precast slabs of ribbed or channel section, the thickness requirement applies to the portion thereof containing the tensile reinforcement.

*The specimen shall be preliminarily loaded at least three times to about half its ultimate strength to eliminate initial strains. The stress-strain data shall be taken with the head of the test machine moving continuously at a uniform speed of from 0.025 to 0.05 in. per min.

410—Holes in webs of joists

(*a*) Holes in the webs of joists shall be formed at the plant, or later drilled (not punched) after approval by and under the supervision of the architect or engineer. For additional requirements see Chapter 8.

411—Extra or concentrated loads on joists

(*a*) Where a joist floor supports partition walls parallel to the joist, or when concentrated loads heavier than the uniform load for which the floor is designed are to be expected, joists may be placed side by side, if adequate provision is made to insure approximately equal loading of all joists.

412—Hollow-core floor unit design

(*a*) Abutting precast hollow-core type units, which are to be completely covered by a field-placed concrete topping slab of controlled strength and placing, Section 403(a), may be installed without falsework or jacking (as in the case of bare units) *provided* there is a continuous controlled strength grout shear key between the units, or a grouted mechanical interlock; and that in designing, the total dead load is assigned to the bare unit, with only the live load considered to be carried by the final topped section of increased depth.

413—Assembled block floor units—Design and tests

(*a*) These assemblies are essentially partially prestressed flexural members formed by post-tensioning, usually with mild steel bars.

(*b*) When it appears that the shear strength of an assembled unit may be less than required, the ultimate strength thereof shall be demonstrated by simple span tests of random sample assemblies to failure over the minimum and maximum spans involved.

(*c*) The strength of the end anchorages of the bars shall meet the requirements of Section 406(a).

(*d*) If topping is used, and relied upon for flexural strength, Section 412(a) for hollow core type units applies. Refer also to Section 301(a) which applies to all precast floor units.

(*e*) Chapters 7 and 8, regarding installation and openings, apply to these units, as to the others.

CHAPTER 5—CURING AND HANDLING

501—Curing

(*a*) Immediately after molding, the precast units shall be cured by one of the methods of Section 501(b), or by keeping them moist at a temperature of 70 F or more for at least 7 days if made of normal portland cement, and for at least 3 days if made of high-early-strength portland cement.

(*b*) Curing by high pressure steam, steam vapor, or other accepted processes may be employed to accelerate the hardening of the concrete and to reduce the time of curing provided, however, that the specified minimum compressive strength is attained, and that its 28-day strength exceeds its earlier strengths.

(*c*) Control cylinders shall be cured under the same conditions as the precast units themselves.

502—Handling

(*a*) Units shall be so stored, transported, and placed that they will not be overstressed or damaged.

CHAPTER 6—MANUFACTURE

601—Workmanship

(*a*) The finished product shall be free of honeycomb or rock pockets. The mix, gradation of the aggregate, and workability shall be such as to insure complete filling of the form and continuous intimate bond between the concrete and all steel.

602—Identification and marking

(*a*) All floor and roof units shall bear a permanent identifying symbol as well as a mark indicating the top of the unit. The identifying symbol shall be the same one used for the unit in the manufacturer's literature and shall be shown in a table on the setting plan together with the length, type, and size of unit and the amount, size, and arrangement of all reinforcement. The tabulated information shall be complete enough to permit the calculation of the load capacity of the unit.

CHAPTER 7—INSTALLATION

701—Installation and construction details

(*a*) A setting plan shall be prepared for every job by the manufacturer for the approval of the architect or engineer. The setting plan shall show the grade of reinforcing steel, the type of aggregate, and the strength of the concrete to be used. If the units are designed as prestressed members, the ultimate tensile strength of the reinforcement, the proportionality limit thereof, and the initial prestress to be used shall also be stated. The plan shall include the locations of all openings, and the headers to be provided.

(*b*) The engineer or architect shall provide a stress sheet which shows the manufacturer the maximum bending moments and shears at the critical sections, except in cases in which the particular span-length unit and its loads fall within the limitations of the load table which has already been approved.

CHAPTER 8—HOLES AND OPENINGS

801—Requirements at structural discontinuities

(*a*) No holes in precast units shall be made on a job site except after the approval by and under the supervision of the architect or engineer. All holes shall be formed, sawed, or drilled, not punched.

(*b*) Holes in the bottom or top faces of hollow units, for conduit or for hangers, shall penetrate the hollow portion of the unit not less than $1\frac{3}{4}$ in. clear from the longitudinal reinforcement.

(*c*) Transverse channeling in the top or bottom faces of units, except over a support, is prohibited. Channels over supports must be so located that the shear strength of the unit is not decreased below that required by this standard.

(*d*) Where a hollow-core (closed section) type of unit is cut, the uniformly distributed load originally to be carried by the cut unit, and by the cutting transferred laterally via continuous grouted and/or close fitting* mechanical keys to adjacent units, may be considered to be uniformly distributed laterally over three identical units on each side thereof, but never over a greater total width than 0.4 of the clear span distance. Cut units carrying partitions or other concentrated loads shall have specially designed reinforced headers or curbs, as specified in Section 801(e).

(*e*) At stairways or other openings when no wall or girder bearing is available for one end of a hollow-core type floor unit, the long dimension of the opening shall be parallel to the length of the unit. Specially designed reinforced headers or curbs at the short side of such openings shall transfer their dead and live load to the units along the side of the opening by devices or means satisfactory to the architect or engineer. Header or curb loads may be assumed as carried by not more than three identical units on each side of the opening, but never by a greater total width than 0.4 of the clear span distance. Such lateral transfer is permitted only when a continuous grouted and/or close fitting* mechanical key exists between adjacent units. In all other cases such loads shall be assumed as carried by the unit immediately adjacent to either side of the opening.

*In some cases the necessary closeness of fit may need to be demonstrated by flexure test.

APPENDIX

Excerpts from "Building Code Requirements for Reinforced Concrete" (ACI 318-56)

The following sections from the ACI Building Code are to be included in the appendix; only their *numbers* and *titles* are listed here in the proposed revision, although any revised standard would carry them in full.

103—Special systems of reinforced concrete
200—Notation
202—Load tests of existing structures
203—Loading and criteria of acceptability
301—Concrete quality
302—Methods for determining strength of concrete
305—Allowable unit stresses in concrete (*Different values of* n *must be used for lightweight concrete.*)
306—Allowable unit stresses in reinforcement
507—Concrete protection for reinforcement

Discussion of this report should reach ACI headquarters in triplicate by Mar. 1, 1958, for publication in the June 1958 JOURNAL.

Title No. 54-25

Failures of Concrete Structures*

By JACOB FELD†

SYNOPSIS

A historical survey of concrete failures of the last half century in the United States; discusses significant examples without attempting to list all failures. Touches on legal aspects from the Code of Hammurabi through English common law with some observations on present code requirements as they relate to different types of failure. Concrete failures are grouped according to their major causes: design deficiency, drafting and detailing errors, concrete mix, supervision omission, frost protection defects, bearing wall deficiency, foundation deficiency, faulty erection techniques, temperature and shrinkage, secondary stresses, and inadequate formwork.

INTRODUCTION

Structural failures, like some medical ills, are unpleasant subjects not to be discussed openly in polite society. That procedure is the best guarantee that the causes and cures will not be discovered and is, therefore, a disservice to the general public and a blot on the engineering profession. Structural failures are herein defined as behavior not in agreement with the expected conditions of stability, of freedom from necessary repairs, and of the desired use for the intended occupancy.

Engineering work involves economy as well as sufficiency—only the few failures which are actual physical collapses become public knowledge, and then only in the relatively small local area interested, if other news does not crowd it off the front pages of popular newspapers. Continuous pressure for greater economy, both in design and construction, has too often resulted in safety reductions to just below the necessary sufficiency, and failure results. Ignorance of the boundary between sufficiency and failure is no excuse; nor is blind compliance with minimal code provisions any guarantee of sufficiency. Accelerated construction programs that expect to rule out the action of gravity by reducing the time during which neutral stability is permitted to exist are no safeguard against instability and failure.

Failures occur in all types of structures, whether framed or wall bearing, whether of timber, steel, or concrete frame. Foundation failures are so common that a structure which behaves exactly as anticipated is hailed as an

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†Member American Concrete Institute, Consulting Engineer, New York, N. Y.

engineering triumph. As structures become more statically complicated, with greater possibility of high secondary stresses from other than the normally computed dead and live loads, greater incidence of failure is noted. As construction procedures are speeded to reduce financing and overhead costs, greater incidence of failure is noted. Well advertised but unproven materials may appear economical and may provide excellent publicity, but a careful survey of the use of new materials shows how few outlast the short period when failures of assembly or material cause such high maintenance and repair costs that further use is recognized as uneconomical. Examples of these materials are the leaking composite wall types, delayed shrinkage of certain lightweight-aggregate masonry block, the crystallization and brittle failure of some metal alloys.

This paper is restricted to concrete constructions, but just as somber a picture can be drawn of other phases of the construction industry—steel bridges and buildings, tanks and silos, dams and bulkheads, footings and foundations of all types and detail.

That failures in structures are not a recent phenomenon is probably best shown by reference to some old regulations. Hammurabi (about 2200 B. C.) included five basic rules in his code of laws, to cover structural failures. The Harper translation reads:

> "If a builder build a house for a man and do not make its construction firm and the house which he has built collapse and cause the death of the owner of the house—that builder shall be put to death.
>
> "If it cause the death of the son of the owner of the house—they shall put to death a son of that builder.
>
> "If it cause the death of a slave of the owner of the house—he shall give to the owner of the house a slave of equal value.
>
> "If it destroy property, he shall restore whatever it destroyed, and because he did not make the house which he built firm and it collapsed, he shall rebuild the house which collapsed at his own expense.
>
> "If a builder build a house for a man and do not make its construction meet the requirements and a wall fall in, that builder shall strengthen the wall at his own expense."

Whether these rules stopped all failures is not known, but they certainly must have been a deterrent to shoddy construction practice and eliminated the possibility of repetitive malpractice.

In the common law developed in England, as is found in the 15th century court records in the reign of Henry IV, the rule was, "if a carpenter undertake to build a house and does it ill, an action will lie against him." Of course, by "ill" is meant "not well" and if all work were done well, there would be no record of failures.

So, historically, the burden rests on the construction industry to see that work is done "not ill" and each partner in the industry, from the architect who conceives the project to the foreman who directs the performance of a part thereof, must lend every effort to avoid and eliminate every possible cause of "ill" structures.

Every new development in the construction industry is accompanied by a concentration of failures. If failures are brought to light and warnings heeded, repetition of failures stemming from the same causes can be prevented. When reinforced concrete was first being accepted as a satisfactory structural material, economy being then as now one of the important arguments in favor of its use, the editors of *Engineering News* in the Apr. 9, 1903, issue wrote:

"One of the special advantages usually claimed for concrete work is that it can be safely built by unskilled labor, but in view of some recent accidents which have occurred it appears well to point out that this principle is not of universal application. For concrete in large masses such as abutments, foundations, retaining walls, etc., there is no doubt that unskilled labor should be employed, under proper supervision. But for certain other classes of work, girders and floors in concrete buildings, it appears that some degree of skilled labor should be employed, or that at least the entire work should be under strict and constant supervision by skilled foremen, architects or engineers. In the carpentry work for forms and falsework especially, there is frequent evidence that the weight of the mass to be supported and the hydrostatic pressure of very wet concrete in columns are not realized by the men who are entrusted with the construction of this part of the work. When concrete has once thoroughly set it will stand very hard service and even overload or abuse, but it should be very strongly impressed upon the men engaged in concrete construction that the wet mass is simply a dead weight to be supported, having absolutely no supporting power in itself. If steel bars, rods, etc., are used, these simply add to the weight of the wet mass.

"The fall of a concrete floor at Chicago, noted in our issue of Dec. 4, seems to have been due entirely to an ignorant man blindly following out instructions which were probably extremely indefinite. He was told to go in and knock out some of the shoring, and he proceeded to knock out every bent, until the unsupported concrete gave way, and in its fall broke through other completed concrete work below.

"Another accident which is said to be due to failure of falsework is recorded elsewhere in this issue. In another case which came under our personal observation a laborer was found knocking away some of the struts and braces under a green concrete floor, simply because (as he told the superintendent when discovered) a carpenter told him to get some lumber. When informed that he stood a good chance of killing himself and other men in the work, as well as wrecking the building, he simply became surly and appeared to think that the superintendent was making a fuss about nothing. The same applies to the concreting gang. The men usually employed for this class of work simply dump the concrete into the forms without discretion, frequently being left without observation by even a foreman for a considerable time. This is especially serious at the junctions of concrete columns and girders. We have heard of instances where forms for girders were filled before the form at the junction with the column was complete, the men simply putting a piece of plank at the end of the form for the girder to make a stop-off and prevent the concrete from running out. If the column is built immediately and the block pulled out there may be little harm done, but there is the liability that the plank may be forgotten, or the concrete of the girder allowed to set before the concrete for the column is deposited. In either of these cases the girder becomes a cantilever, being partially or entirely separated from the column, instead of forming a homogeneous mass with it. There is no doubt that a great deal of concrete work is built by men who are really not competent to undertake it, and that much more work is done without sufficiently strict and continuous expert supervision to ensure the best and safest results. In view of the enormous increase in the use of concrete, and in the variety of purposes for which it is used, it is well for engineers to bear these facts in mind."

This editorial should be required reading for everyone in the concrete industry at least once a year.

In 1918, the American Railway Engineering Association published an editorial article, "Study of Failures of Concrete Structures," with the subheading "A Compilation of Failed Concrete Structures and Lessons to be Drawn Therefrom." The study covers a period of 25 years and classifies the causes under the headings: improper design; poor materials or poor workmanship; premature loading or removal of forms before complete setting; subsidence of foundations, fire, etc. The final conclusion is, "The one thing which these failures conclusively point to is that all good concrete construction should be subjected to rigid inspection. It should be insisted upon that the inspector shall force the contractor to follow out the specifications to the most minute details. He must see that the materials used are proper and are properly mixed and deposited, also that the forms are sufficiently strong and that they are not removed until after the concrete has set. It is believed that only by this kind of inspection is it possible to guard against the failure of concrete structures." So the 1903 editorial advice still held in 1918, and the lesson was not learned and is still not learned.

In 1952, Henry Lossier published a small book, *La Pathologie du Beton Arme*, citing many examples of failures, mostly in concrete frames and special structures, with conclusions similar to those of the 1918 paper plus warnings that errors in the choice of framing and in design details are most important factors explaining failures.

The present paper restricts examples of failures to practice in the United States, citing work of the last 50 years to show repetition of similar conditions. No attempt is made to list all known failures; that would require several volumes of text. Failures have been grouped in the following categories according to their cause: design deficiency; drafting and detailing errors; concrete mix; supervision omission; frost protection failure; bearing wall deficiency; foundation deficiency; erection failures; temperature and shrinkage; secondary stresses; and formwork failures.

DESIGN DEFICIENCY

Deficiency in the basic concrete design, such as amount of reinforcement at points of maximum moment, or dimensions of concrete section to provide compressive and shear resistances under normal loading, is an extremely rare cause of failure. One case that was caught just before placing concrete in forms of roof beams spanning 64 ft over a school auditorium in Yonkers, N. Y., (1925) was where about 4 sq in. of rods had been called for and placed while the design should have required 40 sq in. At the time, girders of this size were most unusual, and the error was picked up by an inquisitive young engineer in the contractors' staff. The girders would have undoubtedly failed if the error had not been corrected.

For simplicity of detailing and rod assembly, it is often expedient to use the same reinforcing for a roof slab as in the typical floor of a multistory building.

This tendency probably explains why in another school building the main reinforcement over an interior support of the floors was incorrectly copied from the design notes and the corresponding reinforcement of the roof design was used. Although this was only one factor of several noncompliances discovered in the investigation, it may have been a considerable influence on the resulting large slab deflections found after the form was removed.

With growing experience in all types of reinforced concrete designs becoming common in all the controls through which a job passes—design, estimating, detailing, field supervision, and construction—the possibility of a gross error in design actually getting into the construction stage is indeed remote. Yet some examples cited under other headings indicate that the possibility exists. Concrete designs need more than a numerical checking; a cursory examination of the design drawings by an experienced engineer or constructor will always show up gross errors in design. It is the small items of underdesign, in the category of secondary stresses from volume or shape change, which, as in structural steel design, are the troublesome difficulties. An arbitrary assumption of the location of points of inflection, a procedure quite popular up to 25 years ago when the free support moments were split as desired into positive and negative factors, often resulted in cracks since the structure refused to accept the assumption if not in compliance with actual relative stiffness of the contiguous members. The reduction in mathematical processes introduced by the Hardy Cross moment distribution procedure eliminates much of this type of local failure, but only when realistic stiffness ratios are determined.

DRAFTING AND DETAILING ERRORS

An actual collapse of a reinforced concrete cantilever retaining wall at Manhasset, N. Y., was caused by a poor drafting job. The design called for 1¼-in. square bars vertically and the draftsman had placed the "1" of the "1¼" directly on a dimension line; actually ¼-in. round smooth wires had been supplied and installed. Total collapse started as soon as the wall was backfilled. It is a strange coincidence that no one questioned the rods delivered and installed. The design had been checked by an outside engineer upon request of the local building department inspector before a permit was issued and the design was approved. Shop drawings of the reinforcement were prepared but there was no record of any checking or approval; yet these shop details were used in the actual construction as a correct guide. The print from which the detailer worked could easily be read to call for ¼-in. rods.

There has been a gradual but persistent tendency for design drawings to show maximum reinforcement conditions and leave to the detailers the decision of where to stop rods, development of laps and splices, and other decisions which should be part of the design. Errors that creep in are easily overlooked in the checking of shop drawings. The European practice, as well as that of many public works projects here, is to detail all the rods on the design sheets. This practice has merit, in spite of the shortage of technical help, since the

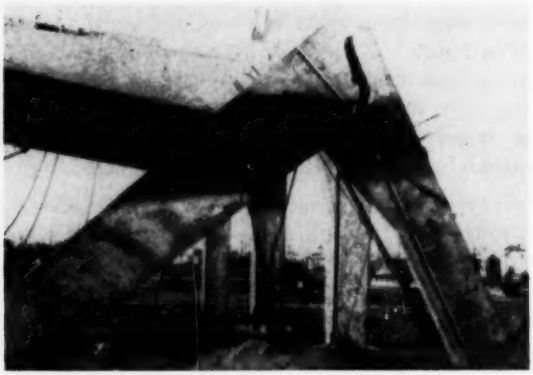

Fig. 1—Collapse of rigid frame bent blamed on poor detailing. Failure followed the unreinforced concrete section of the frame

over-all labor required is actually reduced, even though it means that an office will do fewer jobs in a year.

Does any design office leave to the contractor the job of determining actual dimensions of the concrete work? An editorial in *Engineering News-Record*, Dec. 1, 1955, questioning whether the standards for detailing could be blamed for some of the "concrete structures that have fared badly" brought forth vehement denials of any correlation. It is important that details follow a recognized standard and that everyone use the same symbols and notation. In slabs with straight steel in top and bottom, it has been found expedient and economical to prepare separate detail sheets for the top steel and for the bottom steel. The layers are separately installed and this procedure eliminates all possible errors or misunderstanding.

Collapse of a precast concrete rigid frame bent during construction at the Coronado (California) Naval Base, Feb. 6, 1954, caused an investigation which disclosed insufficient reinforcement at the point of maximum tension (Fig. 1). The splice in the main bars was located at this point and the bars spaced so that there was an open plane with no reinforcement. Permitting a splice at the location of maximum moment had been justified by the effort to reduce costs of shipping fabricated reinforcing bars. All of the 70-ft span frames were scrapped and rebuilt with basically the same design, but the splice was moved 10 ft from the point of maximum tension and the lapped bars welded together.

In large-span frames the concentration of bars makes proper concreting difficult, especially near the splices where the number of bars is doubled. Staggering of splices is often avoided to reduce the number of different length bars. In a large-span concrete frame, where welding was not feasible, the writer detailed the bars to come with upset and threaded ends, and internally threaded steel pipe sleeves were used to splice the bars. The saving in splice material, tie wire labor, and the lower concrete placing costs practically paid for the threading and sleeves.

This problem also is found in the bottom of many multistory columns where the concentration of spliced bars makes it uncertain what concrete encase-

ment can be obtained and often the lower bars are bent inward to provide space for the upper bars. In one project this difficulty was overcome by welding the bars at a level 6 in. above top of footings. Welding cost was no more than the value of the lap lengths saved, but the time factor was serious; welding slowed up the schedule.

As was pointed out by a number of discussions following the report of the Coronado frame failures, every lapped bar introduces shearing stress in the concrete within the splice length. The intensity of the shear depends upon the intensity of the stress in the bars, and the minimum splice lengths specified in codes are not safe if splices are permitted at locations of high tensile or compressive steel stresses.

Bent bars carrying high stress induce a diagonal resultant into the concrete; the amount and direction is easily determined by the requirement of static equilibrium at the point of bend. Many stairway-landing spans show spalled concrete on the underface at the junction of landing and stair soffits where the tension steel is bent with no provision for resisting the outward resultant. Splicing the bars at this point, carrying the tensile steel of the platform and of the stairs separately into the concrete mass, is better detailing practice.

CONCRETE MIX

No collapse failures can be attributed to an improper concrete mix or as noncompliance with the proportions set by a controlled mix design procedure. But practically every report on failure indicates that cores of the hardened concrete are below design strength. When the first Joint Committee code for reinforced concrete was issued for discussion, the writer said in reference to the section on design mix controlled concrete that "he would like to raise the question of what should be done in the case of a building designed and constructed on the assumption that the concrete would test 2000 lb in 28 days and the tests actually showed that the concrete was much weaker."

The answer to this question has been the specified additional curing, the taking of cores for later tests in the hope that the concrete strengths indicated will be sufficient, and finally the making of full scale load tests to prove that the design included too high a factor of safety and, therefore, the weaker concrete provided was sufficient. The true answer is a little more integrity in the furnishing of concrete with the contracted agreement of aggregates, cement, and water, plus or minus other ingredients.

In his own experience the writer has come across some embarrassing situations, probably typical of the experience of other engineers, and a few examples are cited. Where concrete was under extremely careful control and test, the mix contained Type I cement and a field added air-entraining agent; during cement shortages in July, 1955, a carload of Type IA was brought in and used. The result was a concrete with a density of 135 lb per cu ft and a strength of less than 2100 psi at 28 days. The design mix was for 3000 psi and some 20,000 cu yd of concrete had been produced and tested satisfactorily. The low strength concrete came near the end of the job and 170 cu yd of it was delivered

on two separate days. The error was not discovered until some intricate imbedded steel items were encased and acceptance of the job delayed for over a year. Of course, this trouble, assuming that the use of the Type 1A cement was an accident and without knowledge, could have been avoided if the test cylinders were weighed as soon as made. In that way, a mix error can be picked up early and the concrete removed before too much cost is involved. The writer used this simple field check on mix as delivered at the New York Coliseum job and found it a rapid check on plant deviation from design mix, which did occur even with plant inspection.

In two building operations designed by the writer and under construction some 30 miles apart in Texas, at the same time, the concrete design mixes being identical, the cement brand the same and the aggregates from one source, but separate mixing plants, the 28-day tests were consistently higher at one job. The average difference was over 500 psi, not explainable from any variation in slump, temperature, or testing procedures.

On the other hand, on two similar housing projects, on opposite sides of a main road, with concrete coming from a single plant and the truck routed at the entrance to either project as agreed upon by the two contractors, with different testing laboratories, one project showed not a single cylinder below the required 3000 psi, and the other had consistent low tests.

One can only conclude from such experiences that the cylinder test data is indicative but not conclusive, and no single correlation between failures and mix production is to be expected.

SUPERVISION OMISSION

Competent and strict, almost unfriendly, supervision seems to be one key to the problem of how to prevent failures. Again from the writer's early experience in concrete construction are cited some examples of what can occur if supervision is lacking. Although the descriptions may sound fantastic, the four examples are correct reports of actual occurrences documented by photographs.

The first was a buttress-type retaining wall where the foreman insisted that the horizontal bars shown bent over the supporting buttress should be cut and hooked into the buttress. The plan was perfectly clear and the bars had come to the job prefabricated and correct. This wall was adjacent to a railroad right-of-way to provide support for a siding. It required a call from the railroad bridge engineer with the threat that no siding track would be built to get the reinforcement corrected.

The second also concerned a retaining wall, this one a cantilever type. The footing had been built and the dowels for the stem were being bent out of position and the reinforcement for the wall proper was being placed along the front or exposed face when a routine inspection stopped the work. The foreman in charge, an old experienced concrete laborer, insisted that the plan was wrong and he was "making a fix." As he argued, the cantilever wall had two

faces, the back face was held by the earth fill, the front face also needed support and that is where he was placing the bars. He was replaced.

The third case was a supported slab where concreting was specified to follow inspection and approval of reinforcement in place. At the arranged date for inspection, the slab bars were all in place but the beam bars had not yet arrived at the job. On the next day, without appointment, the inspection trip was repeated (the project was a three-tier concrete home some 30 miles from the office) only to find the concrete already completed and a truckload of beam bars just arrived. It took considerable argument to have a strip of beam bottom form removed and the still soft concrete chopped out to see the beam bars. They were all ⅜ in. and the contractor had merely substituted twin ⅜-in. bars for each ¾-in. or larger bar required. It was only after a threat of calling the police was followed by an actual trip toward the station, that the contractor agreed to immediately demolish the floor and rebuild it—which was done.

The fourth case is that of a group of garden apartment houses founded on wood piles. A small percentage of the walls almost immediately after completion showed serious distress and settlement cracks. Considerable time and money were spent on new borings and soil studies. Piles exposed and jacked showed completely satisfactory resistance. Exposure of the concrete grade beam immediately gave the explanation. The top of some pile caps had been installed below plan elevations and the grade beam built exactly as per plan, resting on a layer of backfill *above* the pile cap.

In various investigations of structural distress, the writer has found some similar sloppy and shoddy work in masonry, in steel, and in timber. How can it be prevented without competent supervision?

The example of the retaining wall with ¼-in. bars described above could never occur if just one competent person were at the job. The warning of the 1903 editorial in *Engineering News* should be seriously taught and learned.

One of a series of failures which brought on the editorial warning was the collapse of the three-story department store in Corning, N. Y., in 1903. Collapse occurred just after completion. Floors and walls were of concrete, the reinforcement being steel wire-mesh fabric. The architects specified load tests as a measure of acceptance, 400 psf on the ground floor and 200 psf on each upper floor, over areas 5 x 5 ft, two selected areas on each floor. The architects complained that rumors had reached them to indicate insufficient reinforcement placed in the floor slabs. The ground floor passed the test, but the loading of the upper floors resulted in collapse of the rear portion. Inspection of the wreckage confirmed the architects' complaint that some reinforcement had been omitted.

There is one case of a three-story military school building which has cracked into short segments from temperature and vibration causes. But one extremely mysterious factor in the investigation was the complete omission of bars in some ribs of the waffle slab, although shown on both the design and detail plans. Defects strangely enough are no worse in the areas where rods were

omitted than where provided as required by the design. This structure was built under reportedly the most careful inspection and supervision. Yet, it must be concluded that some floor areas were concreted without an inspection of the reinforcement.

FROST PROTECTION FAILURE

Concrete can be safely installed in freezing weather, if precautions and frost protection equipment are prepared and available before work is started. Witness the great number of successful winter operations in Canada, where contractors have learned to live with the cold weather and not hope for the thaw that does not come at the right time.

There have been some serious concrete failures caused entirely by lack of proper methods and protection. The collapse of the concrete bath house at Atlantic City, N. J., in March 1906, was attributed almost entirely to frost penetration into the wet concrete, resulting in separations between beams and columns and between slab and girders.

A more spectacular frost damage collapse was the seven stories of a proposed eight-story concrete hotel building at Benton Harbor, Mich., Jan. 28, 1924. Most of the concrete in the debris could be just shoveled away; only a few of the lower story columns and first floor beams remained intact. Concreting was in December and January. Although no one was injured in this collapse, every detail of specification compliance was checked and, as far as could be determined, the ingredients and the mix were correct, the aggregates were thawed before mixing, and the water in which some calcium chloride was dissolved was heated. Tarpaulins were available for one story only. Temperature records showed some below freezing days in December and practically not a single day without freezing in January. Compressive strengths of five samples taken from the debris ran from 247 to 718 psi, not enough to hold the dead weight.

A common frost damage on structures results from the practice of placing "mud-sills" on frozen ground which softens from the drip and the frost protection heaters, causing unwanted sags in the supported structure. Another serious frost damage occurred in a substructure contract where completed piers with anchor bolts imbedded were left exposed during the winter. Bolts had been placed in oversize sleeves to permit some adjustment to fit steel base plates. Water filled the sleeves, froze, and cracked off the edges of many piers. Such open sleeves, even if only for future railings, should be covered or filled with nonfreezing materials.

BEARING WALL DEFICIENCY

That concrete is a rigid material and therefore will transfer moments into supporting walls when the beams or slabs are imbedded should be a recognized fact. Yet, bearing walls are not designed to resist these support moments and failures result. The failure of a concrete floor under test at Trenton, N. J.,

in 1903 (which seemed to be a bad year for concrete failures) is one example. This was in a completed mill building, 50 x 300 ft, four stories high. The floors consisted of two spans, resting on the exterior walls and on a center girder supported by columns. A 4½-in. slab spanned between 20-in. deep beams which were imbedded 12 in. in the brick walls. A test load on the third floor covered the width of the building for a length of 16.5 ft. The beams pulled out of the wall which showed a slight buckle inwards and the floor failed and broke through the second floor.

The 1956 roof collapse of a one-story school in Orlando, Fla., is a similar example of insufficient bearing wall resistance. Placing 56-ft precast beam spans on an 8-in. block wall provided no resistance to the rotational force of the deflected beams. The committee report recognized this fact but expressed it in language which tends to hide the conclusion: "The combined effect of this water" (from a heavy rainfall) "induced excessive deflections progressively and thereby greater loads on the prestressed concrete double-tee slabs, with commensurate lateral pressure on the walls." The walls were of 8-in. block, 14 ft high without pilasters, and with window openings in over half the lengths.

FOUNDATION DEFICIENCY

Because structural continuity almost always exists in concrete structures, they are more vulnerable to damage from foundation deficiency than steel or timber structures. Even partial loss of one support causes radical revision of moment and shear stresses in a continuous structure (Fig. 2). Seldom is a structure designed for such a condition. Exceptions are continuous bridges and decking structures to support street loading over subway and similar excavations. Typical custom in designing decking for New York subway construction, where main beams are carried over several supports, some of which may accidentally be displaced or clearance requirements make it mandatory to move one support at a time, is to permit normal stresses with all supports in place and survival stresses with any one support removed.

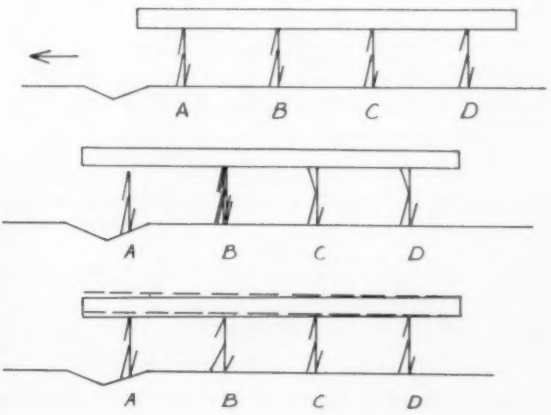

Fig. 2—Load transfer from settlement of support of rigid structure. Uniform reactions at *A*, *B*, *C*, and *D* are completely modified if *A* is at a soft spot, causing *B* to more than double, and changing *D* from compression to uplift

To follow such procedure in all structures would considerably increase construction costs. It is therefore important to provide uniformly consolidating supports for all columns of a concrete structure. Steel structures in New York have shown partial collapse where unequal bearing is accidentally provided. Three such occurrences are described in another paper by the writer.[15]

A recent well publicized collapse of a four-story flat slab concrete structure during the placing of concrete in the fourth floor slab was diagnosed on the basis of personal observation and photographs which all showed a characteristic funnel shaped debris pattern. It has been found in the investigations of several collapses, that the debris leaves such a funnel (Fig. 3) and that the axis of the funnel points toward the first movement. The axis pointed toward one column. In addition, it was found that a plywood scale model of the portions which had collapsed on all the four floors, when suspended upside down, balanced at the same column. Later investigation, after removal of the debris, showed that this column in the bottom story was tipped toward the exterior wall and the foundation conditions under the exterior wall columns

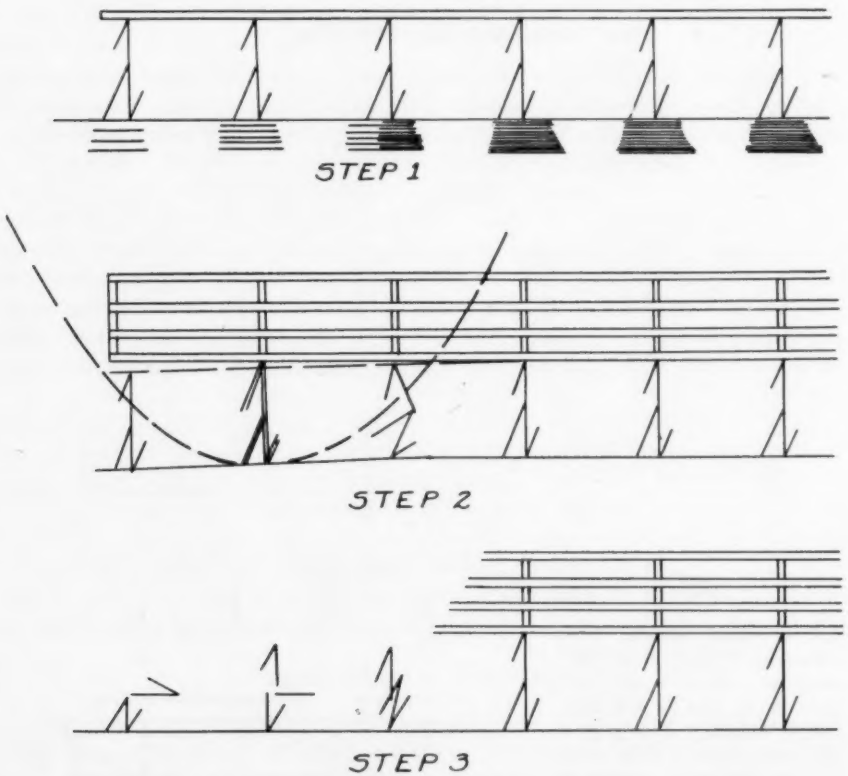

Fig. 3—Steps in failure of concrete frame with some columns on poor support

Fig. 4—Collapse of a concrete flat slab structure

were not the sound rock called for on the design plans. Transfer of load, in this case the weight of the concrete alone, caused overstress at foundation level, tipping and loss of support in the flat slab floors. The slab could not span 54 ft, after one support became inoperative, and so collapse resulted (Fig. 4). The investigation showed several other discrepancies between design assumptions and actual performance, and the structure was almost completely demolished before reconstruction after some 40 column footings were reinforced or enlarged. First publicity on this collapse placed the blame on form support failure; by the time the facts were available, this matter no longer was considered news. Lack of control and inspection has cost everyone concerned tremendous sums of money, not to mention the loss of three lives and injury to other workmen.

Grain elevators are especially vulnerable to foundation insufficiency, the varying loading as bins are filled increasing this difficulty. Classic example is the Canadian grain elevator which tipped in 1914 and was later righted. A concrete grain elevator in Fargo, N. D., failed without warning in 1955, and the final diagnosis was the settlement of the footing on one side. The structure had been in use about a year and included 20 silos 19 ft in diameter and 120 ft high.

Concrete bridge abutments should not be built monolithic with their wing walls. The considerably unequal base pressures, since the wing walls receive no live loadings, causes unsightly separations and sometimes tipping. It is best to consider the abutment as one structure and the wing walls as adjacent but separated retaining walls.

ERECTION FAILURES

The lifting of a prefabricated structural unit, whether a precast pile, the suspended span of a cantilever bridge, a precast or prestressed or plain reinforced beam, or a lift-slab section can only be safely accomplished if erection stresses do not go beyond the yield points and the lifting equipment provides sufficient and uniform supports. Failures in this class are always insufficiency of small details or nonuniform action of the lifting hardware. Small horizontal forces will upset the neutral equilibrium existing during upward lifting of a large mass unrestrained laterally.

Wide acceptance of the recently developed lift-slab technique has been delayed somewhat by several such erection failures. There were three successive failures on three separate attempts to complete a lifted roof slab section at Ojus, Fla., (near Miami) in 1952. Only the last was attributed to any structural design deficiency. The first failure was in a 5-in. pipe stub supporting a jack on top of the permanent column; the pipe buckled and dropped the jack, while eccentricity in lifting was being adjusted. The second failure was triggered by stripping of defective threads on the inverted flange supporting the slab. The third failure was a shearing in the edge of the concrete cap above the column; the computed unit shear was 93 psi over the circular concrete section. The final and successful design included some extensions of reinforcement and additional steel shear resistance.

During a demonstration erection of a lift slab in San Mateo, Calif., in 1954, while guys were being adjusted to counteract a 3-in. leaning of the columns, the 65 x 70-ft slab unit drifted laterally 15 ft and fell. Six workmen and four of the 60 spectators were injured and recently (November, 1956) were awarded damages of over $100,000 as a result of litigation in a California court. The verdict was against the architect, general contractor, and lift-slab contractor. Fortunately, most of the visitors were cleared off the slab. Failure was partly explained as the result of eccentric loading when the observers were concentrated on one side of the slab.

During a wind storm with possible gusts of 35 to 50 mph—normal late spring weather in Cleveland—an eight-tier lift slab had been completely raised, but before the columns had been welded to the slab collars (Apr. 7, 1956) it drifted laterally almost 7 ft out of plumb. Emergency action stopped further movement; cables anchored to everything available in the vicinity held the structure until permanent measures were taken. In spite of a definite order from the building commissioner to demolish the structure above foundation level, the frame was actually straightened by jacking diagonally across the first story, after placing temporary shores at each column for the full height of the building. The incident clearly indicates the necessity for careful analysis of stresses and possible deformations during erection.

TEMPERATURE AND SHRINKAGE

Although few examples of actual collapse can be blamed on temperature and shrinkage stresses, considerable cracking and spalling in concrete structures results from lack of freedom to modify dimensions with temperature changes and from aging. Unfortunately, due to the thermal insulation afforded by the concrete to the inner volumes and to the nonuniform rate of shrinkages in elements of different thickness and shape, dimension changes are not entirely linear. Rotational displacements result and seriously affect brittle masonry surfacing, windows, and door frames. Although the addition of uniformly distributed and continuous reinforcement is recognized as a proper resistance against thermal changes, the apparent distress from shrinkage is not affected by such steel. Actually, shrinkage distress is more likely with larger amounts of reinforcement because the steel will not shrink and internal bond failures must result.

Temperature restraint reinforcement is always specified for floor and roof slabs. But it is seldom required for beams and girders. Even the minimum 0.25 percent requirement, if applied to all concrete sections would prevent possible shear failure of totally unreinforced surfaces. The older literature suggests a minimum of 0.3 percent of the cross section as necessary to prevent cracking from shrinkage and temperature. This amount has not always prevented the formation of cracks and Dreyer (1933) stated that the experience of the Pacific Gas and Electric Co. indicated 0.65 percent necessary.

The differential shrinkage between thick and thin reinforced concrete members in intimate contact can cause an eccentric pull on the members; this is the usual reason for the cracks forming in slabs connecting to heavy girders. R. F. Blanks presented some data (1951) on this subject and showed the results of tests by Pickett in measuring the coefficient of shrinkage at 9 months for various thicknesses:

1½ in. thick:	720 millionths
3 in. thick:	700 millionths
6 in. thick:	580 millionths
12 in. thick:	330 millionths

In addition to such shrinkage strains which bring in large tensile stresses within a concrete mass, there is the change in length from temperature differences. The average coefficient for reinforced concrete is 0.0000055 per deg F change. For an elastic modulus of 5,000,000 psi, each degree change induces a stress of 27.5 psi. If no reinforcement is provided across the section, it takes only a few degrees temperature change to result in stresses in excess of the maximum tensile resistance of plain concrete.

Some of the aging of concrete as seen in continuous increase of cracking may possibly be explained by a difference in thermal coefficients of expansion and contraction. Vetter (1933) quotes from Deutsche Ausschuss fur Eisenbeton *Bulletin* No. 23 that for dry concrete specimens 90 days old, the average coefficient for expansion is 0.0000069 and for contraction is 0.0000056. If

this can be substantiated, it would explain the tendency of concrete structures to grow, with cracks forming to make up the differential in lengths, even where aggregate reactions are not a factor in volume change. A length of 400 ft would show residual shrinkage cracks of 0.31 in. after a 50 deg F temperature fluctuation.

Air force warehouse failures

The immensity of thermal and shrinkage stresses as compared to live load stresses is shown by failures of the warehouse roof rigid frames at Wilkins Air Force Base, Shelby, Ohio, on Aug. 17, 1955, and at Robins Air Force Base, Macon, Ga., on Sept. 5, 1956. Both failures as well as the experimental and theoretical studies made in the attempt to evaluate the forces which could cause such failures and the details of resistances which must be provided have opened up a healthy discussion in the technical press. At least it is admitted that the real causes are not quantitatively known. Similar warehouses were constructed in many other locations, all 400 ft wide and of various lengths, but always in 200-ft units with complete joints between units. The rigid frame spans are on 33 ft spacing, and consist of six 67-ft continuous spans, the third one from one end having an expansion joint.

Standard design used for preparation of contract drawings fixes the main girders but leaves open the choice of subframing in the 33-ft spans and of the actual roof construction. In the various locations, a great variety of roof construction was used, from precast deep channel sections to span the 33 ft to prestressed joists set into the forms of the girders and so anchored in, with cast-in-place gypsum roof slabs. Also, a great variety of cements and of aggregates were used in the several locations. Yet, some girders in almost every structure show some signs of distress, more so with the early designs which did not require continuous reinforcement in top and bottom of the girders and more so with the designs having fewer stirrups. No amount of check computations has explained the causes; modifications of the standard code were suggested and in part approved. A considerable test program was carried on at the Portland Cement Association laboratory; model beams reproduced the various designs used and the tests to failure proved that under the design loadings, no failure or even cracking was possible. Yet, failures did occur at two locations and cracks at many, with practically no live loads.

Observations at several installations in geographically different parts of the country indicate the following pertinent data which must be considered in any logical explanation of what happened and how to avoid it.

1. The expansion joints do not operate, and the 400-ft length is a single unit as far as thermal and shrinkage stresses are concerned.
2. Both collapses came at about the date of maximum net heat absorption gain, the night loss becoming less than the day gain. The 1955 failure was after an unusually hot spell.
3. Identical designs built at the same location and approximately over the same time period, with the same aggregates do not show the same amount of cracking; the smaller unit built at a much slower rate shows much less distress.

4. Rotational strain of the columns about a vertical axis and of the girders at cracked sections have been found. Except where caulking was used in place of cement mortar in the joint between columns and exterior block walls, spalling and horizontal cracking of the blocks is common. The side faces of the girders across a serious crack are not a continuous plane.

5. Failure of the models would only occur under loadings below the dead plus live load when an axial force was applied to the girder model. With an axial force of 200 psi on the minimum cross section, the ultimate load capacity of the model was substantially the equivalent of dead load only.

6. Although some cracks appeared during the construction period, most of the distress was not found until the frames were almost 2 years old.

The final reports and conclusions have not yet been prepared. Certainly, some lessons can be learned:

1. Just as it takes time to make good bread, it takes time to make good concrete; a rush job is vulnerable.

2. Expansion joints only serve their purpose when they operate, and frozen joint details are worse than no joints.

3. Lateral restraint of all members is assumed in our basic designs; such restraint must be provided, especially at the compression faces of continuous structures.

4. If torsional strain can absorb expansion with less internal work than axial expansion plus bending of the supporting columns, then the girders will deflect laterally in a sine curve pattern and rotate the columns; cracks must open if the girder is not braced or reinforced to compensate for such torsional strains.

In 1956, bridge buckling in Queens, N. Y., was found to be a similar lateral distortion of the fascia wall and parapet on a rigid frame overpass. The distortion provided the extra length required by thermal expansion with less internal work than an axial elongation of the unbraced parapet.

SECONDARY STRESSES

Under the general heading of secondary stresses are those cases where direct stress computations from normal loading would indicate no reason for cracking, but the resulting strains induce stresses along other axes which will give trouble. Such conditions always exist, but the intensity in normal designs is not high enough to overcome disregarded internal strengths such as the tensile values of concrete.

Structures like flat slabs transmit loadings to supports by the easiest way, which may not be the pattern assumed by the design. Cracks then result as the frame becomes hinged and tries to conform to the assumed pattern. Slab loads are assumed to travel to the columns by bending of the allocated bands. Actually, the spandrels, due to the unbalanced bending moments transmitted to them, rotate and carry the reactions partly by torsion. Diagonal torsion cracks then appear in the face of the spandrel section, especially if a deep beam is used. Such rotation has been known to push the masonry facing out of position and form horizontal cracks in the mortar jointing, with possible rain infiltration into the building.

Similar torsional action causes corners of roof slabs to curl and distort roof flashing and to crack exterior faces of exterior columns.

Storage bins designed for wall tension as a resistance to internal grain or liquid pressure must be free to expand at the contact with rigid base slabs or foundation. As Mensch pointed out, the tank bottoms are highly vulnerable to failures, even if the shell is prestressed. Similar trouble must be designed against at the junction between adjacent bins whether circular or rectangular, as is described by Vandegrift.

A similar action explained the diagonal cracking in a large prestressed concrete pressure main near the joints which were grouted into precast concrete sleeves. The expansion of the pipe was not permitted near the ends by the rigid sleeves. The trouble disappeared when the joints were caulked with elastic material in place of the cement grout.

Considerable experience is being gained by the detailed description and diagnosis of failures in prestressed concrete. An interesting case was the explosive disintegration of precast beams in Wellington, New Zealand, in 1955. Prestressed beams 105 ft long were being shipped from the plant when careless hauling caused a beam to tip and rotate. The beam virtually exploded under release of prestress and in so doing touched off another beam which was standing erect. It also broke into fragments.

Proper resistance for diagonal resultants of bent wires is necessary. Bent wires must be of such composition that surface defects or unbalanced stress on the cross section will not cause "stress corrosion" failures. Such brittle failure breaks are known to have required the replacement of cables in two suspension bridges, the Detroit-Windsor and the Mt. Hope spans. Similar failures have been reported for tanks including the Richmond, Calif., reservoir in 1955, and in some continuous prestressed bridges in Europe. Considerable information on this subject is found in reports from the International Prestressed Concrete Conferences at Ghent in 1951 and Amsterdam in 1955.

Combined high bending and shear stresses require some special design precautions, this being a special case of the condition of combined tensile and compressive stresses. Only such approach can explain the cracking of long prestressed members and the failure of the prestressed Vierendeel truss at Lodi, Calif., in 1953.

The recent collapse of the four-story flat slab office building in Jackson, Mich., during construction, is attributed to shear failure around the columns at places where high compressive stresses must be also resisted. Special bracket details were designed for such shear resistance and top reinforcement was welded to the top plates of these brackets in the design of the New York Coliseum. Bypassing columns with the top band of steel is not safe practice.

FORMWORK FAILURES

So many formwork failures are reported that space permits only a summary of the types of failure and, unfortunately, only seldom is the real cause divulged. It does seem, however, that the pattern is fairly uniform: fast rate of placing the concrete and failure during the stage of construction when forms

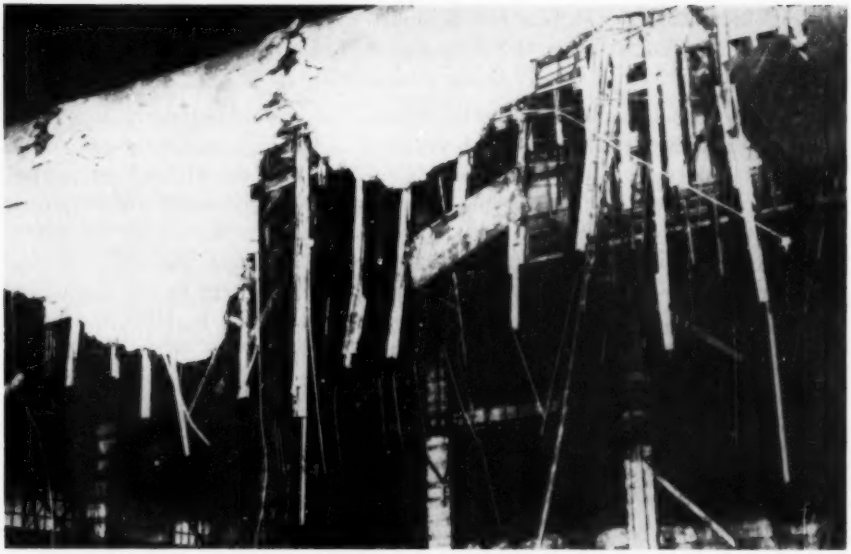

Fig. 5—Form failure at Welland, Ontario, November, 1942

have been used for several times and work is routine. No failures are reported at the beginning of the jobs; seldom in small operations.

Formwork failures date back to the beginnings of reinforced concrete. The fourth floor of the Chicago Apartments in 1902 collapsed when a careless or misdirected worker removed all shoring with the forms, and the concrete fell right down to the basement. In 1903, there were formwork failures in Pittsburgh where a sixth-floor slab fell through to the basement and in Milwaukee where a part of the fourth floor fell and broke through the floor below. Both failures occurred just at completion of the day's work.

The reservoir roof formwork collapse at Santa Monica, Calif., in 1949 was the first of a series of failures of high falsework made up of double-tier shoring. The photographic record of the debris is so similar to the 1955 formwork failure at the New York Coliseum that a copy of the picture was accepted as that of the Coliseum case at a conference held only a few days after the latter incident. An unofficial theory of the cause of the Santa Monica failure was that a workman struck supporting posts with a wheelbarrow and started a chain reaction of collapse.

A similar formwork failure at the Washington, D. C., reservoir roof in 1941, the shores consisting of double-tier wood posts, was ultimately charged to sabotage. In 1949, another double-tier formwork at the Parke-Bernet Galleries, New York, failed under the action of power buggy concrete placing. No explanation of the failure was reported. The supports were a combination of steel pipe adjustable shores supported on timber posts.

In 1953, another similar failure was reported on the Everett, Mass., concrete tank roof. Placing of concrete was at the rate of about 100 cu yd per hr and the work was within 10 min of being completed.

In 1955, forms for the Kentucky State Fair grounds stadium supported by open web joists on 2-in. pipe shores 16 to 18 ft high suddenly gave way, and no explanation has been reported. Also, the upper exhibition floor level at the New York Coliseum, on double-tier shoring, the bottom of wood framing and the top of steel pipe, with 700 cu yd placed in less than 7 hr, started to weave, rotated and fell 22 ft to the lower floor. The area that fell was supported on about 1000 shores; the debris fall did no damage to the main floor which was checked by surveys and full scale load tests. In 1956, there was another formwork failure in Jackson, Miss.

With concrete placing rates of some 100 cu yd per hr and the use of power buggies with good brakes stopping a 2500-lb load "on a button," formwork design must take into consideration the horizontal and possible synchronous impacts from such equipment. Forms are supported on continuous framework and, like all continuously supported structures, must have uniformly fixed reactions. As the plastic concrete covers the forms, reactions on the posts change and there is good possibility of uplift at posts beyond the area covered by the concrete. Careful nailing for uplift resistance must be provided. Wedging posts to level up forms are operations which must be done under careful supervision, to avoid leaving posts disconnected from the deck. This would provide a possible beginning of failure if the reaction thereon became negative. How to obtain sufficient and capable direction for formwork erectors to avoid all possible trouble is the present problem. No amount of inspection can replace that direction.

CONCLUSION

Finally, it should be pointed out that failures can be avoided if concrete is not used where vulnerable to known difficulties, such as exposure to sea water unless entirely dense and imprevious; exposure to lactic acid resulting from stagnant wastes in milk, beer, and meat processing plants; exposure to ammonia fumes where cinder concrete is specified; even exposure to sawdust wastes has been found detrimental.

Structural failures have occurred at all times and in all types of work. Reasons for collapse are often determined after long investigation, but usually after interest in the incident has waned, and little publicity is given to the final report. Cause of minor incidents is seldom announced. As a result, recurrence of similar accidents and failures is too common. It is hoped that publicity of all failures will become more common and both the engineering profession and the construction industry can learn by the mistakes of others and avoid such incidents in their future work.

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Title No. 54-26

Ultimate Strength Design Charts for Columns Controlled by Tension*

By TUNG AU†

SYNOPSIS

Basic design equations for eccentrically loaded concrete columns in this paper are based on the report of the joint ACI-ASCE committee on ultimate strength design.[1] To simplify the design procedure for columns with symmetrical reinforcement or with round cores, design charts were provided in a paper by Whitney and Cohen.[2] While these charts are applicable to eccentrically loaded columns which may be controlled either by tension or compression, it seems that they are not entirely satisfactory for columns controlled by tension due to the close spacing of e'/t lines in this range. To meet such need, this paper examines certain omissions in the ACI-ASCE committee report, and presents a new set of design charts for rectangular sections with symmetrical reinforcement and for square and circular sections with spiral reinforcement which are subjected to combined bending and axial loads and controlled by tension.

INTRODUCTION

The method of designing reinforced concrete columns has followed a zigzag course during the past 50 years. The original idea was based on the ultimate load carrying capacity of the members. When the elastic theory was adopted for the design of flexural members, it seemed consistent to accept the same theory for the design of compression members. As the shortcomings of the elastic theory were gradually recognized, the ultimate strength method was again introduced, although on a limited basis, for the design of members subject to axial loads only. Thus, the method used to design members subjected to combined bending and axial loads is an arbitrary combination of both theories and is perhaps the most unsatisfactory provision in the design codes. The reasons for introducing ultimate strength design for all types of members have been expounded eloquently by many researchers and engineers and are embodied in the "Report of ASCE-ACI Joint Committee on Ultimate Strength Design" (hereinafter referred to as the ACI-ASCE report).[1] The latest edition of the ACI Building Code[3] has adopted the ultimate strength method as an alternate design procedure.

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Although the ultimate strength method is no more complicated than the elastic theory, the formulas for column design in the ACI-ASCE report appear long and cumbersome. Furthermore, there are some inconsistencies and omissions in the formulas for eccentrically loaded columns of square or circular section with round cores. A paper by Whitney and Cohen[2] furnishes some missing links, and the author has pointed out several others.[4] Whitney and Cohen also provide design charts in the form of interaction curves for columns with symmetrical reinforcement or with round cores subject to eccentric loading which may be controlled either by tension or compression. However, since the spacing of e'/t lines in these charts is very close in the range controlled by tension, this paper presents another set of design charts which are believed to be more suitable to cover such range. These charts are, in a way, similar to the diagram for rectangular members prepared by the Portland Cement Association and recommended as Fig. 6 in the ACI-ASCE report. However, in this paper, eccentricity e' instead of e has been used and the overall depth t has replaced the effective depth d in the process of computing concrete area and percentage of reinforcement in the case of rectangular columns with symmetrical reinforcement. For square and circular columns with spiral reinforcement, similar variables are used, and the limiting loads are derived on basis of the formula for the limiting load of rectangular column in the ACI-ASCE report.

DESIGN EQUATIONS

Notation

The letter symbols used herein conform with the definitions of ACI Building Code wherever possible.

A_g = gross area of section
A_{st} = total area of longitudinal reinforcement
A = compressive area in circular section subject to eccentric loading
a = depth of rectangular stress block in rectangular or square section
b = width of rectangular section
d = distance from extreme compressive fiber to centroid of tensile reinforcement
d' = distance from extreme compressive fiber to centroid of compressive reinforcement
D = total diameter of circular section
D_s = diameter of circle passing through the center of longitudinal steel bars forming the core in square or circular sections*
e = eccentricity of axial load measured from centroid of tensile reinforcement
e' = eccentricity of axial load measured from the center of the section for columns with symmetrical reinforcement
f_c' = 28-day cylinder strength
f_y = yield point of reinforcement, not to be taken greater than 60,000 psi
m = $f_y/0.85f_c'$
p_t = A_{st}/A_g
P_b = limiting load for balanced design which is equally likely to cause failure in tension or in compression
P_u = ultimate strength of eccentrically loaded member
t = total depth of rectangular section or one side of square section
t_s = $(d - d')$ for rectangular section with symmetrical reinforcement

*This is adapted from the notation in the appendix of the ACI Building Code. In the report of the ACI-ASCE joint committee, the term d is used to represent this quantity as well as effective depth. See References 1 and 3 for details.

Basic concept

The ultimate strength of members subject to combined bending and axial loads can be computed from the equations of equilibrium of forces and of moments. When the ultimate capacity P_u obtained from these equations is less than the limiting load P_b, the section is controlled by tension; otherwise it is controlled by compression.

The basic assumptions for the derivation of equations for eccentrically loaded columns in the ACI-ASCE report are used. It is further assumed that the concrete area displaced by compressive steel is neglected since such an assumption will cause small error in sections controlled by tension.

Rectangular columns with symmetrical reinforcement

For a rectangular section with symmetrical reinforcement (Fig. 1), half of the steel is effective on each side of the section. With $(d - d') = t_s$ and $d = \frac{1}{2}(t + t_s)$, the following equations of equilibrium are obtained from the ACI-ASCE report:

$$P_u = 0.85 f_c' ba$$

$$P_u e' = 0.425 f_c' ba (t - a) + 0.5 A_{st} f_y t_s$$

Eliminating the term a from the equations and expressing the result in dimensionless form, it follows:

$$\frac{P_u e'}{f_c' bt^2} = 0.5 \left(\frac{P_u}{f_c' bt}\right) - 0.589 \left(\frac{P_u}{f_c' bt}\right)^2 + 0.425 P_t m \frac{t_s}{t} \quad \ldots\ldots\ldots (1)$$

When the section is controlled by tension, Eq. (1) can be used to determine P_u, algebraically if necessary, but more suitably by graphs in this case. The limiting load can also be derived from the ACI-ASCE report and expressed in dimensionless form:

$$\frac{P_u}{f_c' bt} \leqq \frac{P_b}{f_c' bt} = 0.36 \left(\frac{90{,}000}{90{,}000 + f_y}\right)\left(1 + \frac{t_s}{t}\right) \quad \ldots\ldots\ldots\ldots (2)$$

When the section is controlled by compression, a semi-empirical formula is suggested by the ACI-ASCE report and can be represented by the following equation:

$$\frac{P_u}{f_c' bt} = \frac{0.85 p_t m}{\dfrac{2e'}{t_s} + 1} + \frac{1}{\dfrac{12 t e'}{(t + t_s)^2} + 1.18} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The relationship of the variables in Eq. (1) can be represented graphically by a chart with $p_t m t_s/t$ as ordinate and $P_u/f_c' bt$ as abscissa while considering the term $P_u e'/f_c' bt^2$ as parameter. Eq. (2) will define the limit of application for Eq. (1) or the chart derived therefrom. Since Eq. (3) will not appear in

the chart for sections controlled by tension, this paper will not discuss further the semi-empirical nature of the equation. It will suffice to indicate here how Eq. (3) eventually leads to similar equations for sections with round cores.

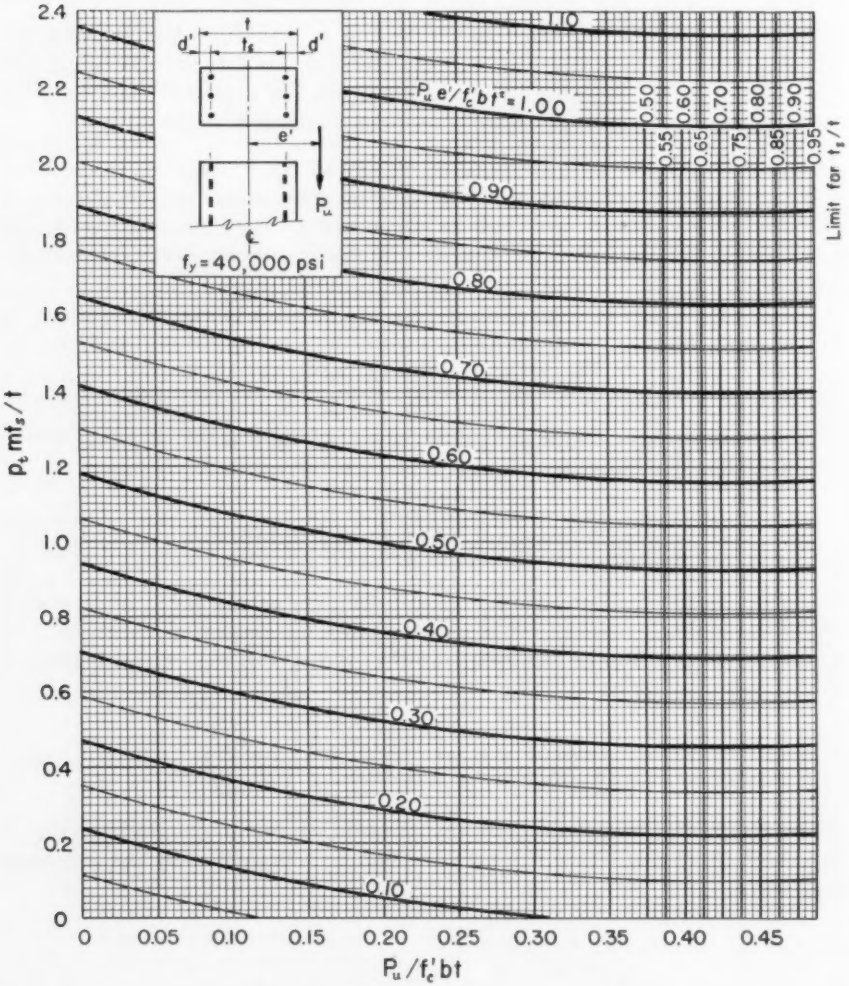

Fig. 1—Rectangular column with symmetrical reinforcement controlled by tension*

*A set of nine design charts including Fig. 1, 2, and 3, plus similar charts for f_y = 50,000 psi and 60,000 psi, is available in large size (about 12 x 12 in.) from ACI headquarters at $2.25 per set.

Square columns with round cores

For a square section with round core (Fig. 2), the ACI-ASCE report assumes that half of the steel is effective on each side of the section and that the effective $(d - d')$ equals $0.67D_s$. This is applicable to sections controlled by

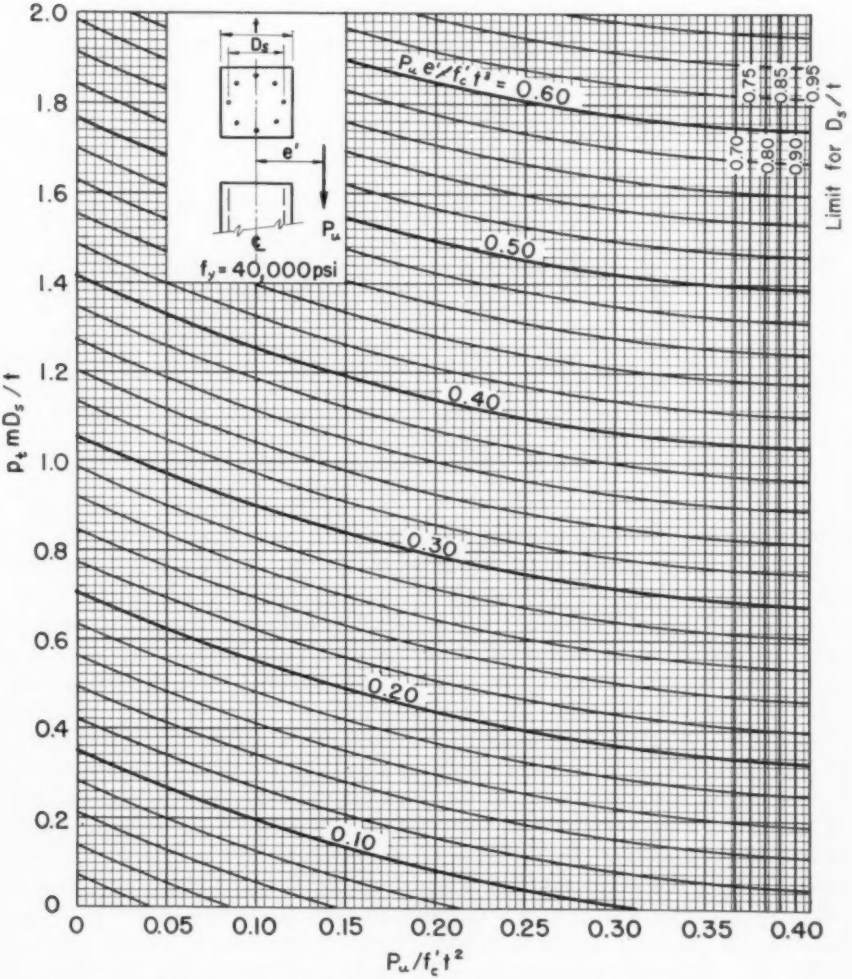

Fig. 2—Square column with spiral reinforcement controlled by tension*

*A set of nine design charts including Fig. 1, 2, and 3, plus similar charts for f_y = 50,000 psi and 60,000 psi, is available in large size (about 12 x 12 in.) from ACI headquarters at $2.25 per set.

compression as well as by tension. Thus, the equations of equilibrium for square sections with round core can be expressed as follows:

$$P_u = 0.85 f_c' t a$$

$$P_u e' = 0.425 f_c' ta (t - a) + 0.5 A_{st} f_y (0.67 D_s)$$

Hence, by eliminating the term a, the equation for section controlled by tension becomes:

$$\frac{P_u e'}{f_c' t^3} = 0.5 \left(\frac{P_u}{f_c' t^2}\right) - 0.589 \left(\frac{P_u}{f_c' t^2}\right)^2 + 0.283 p_t m \frac{D_s}{t} \quad \ldots\ldots (4)$$

For different ratios of D_s/t, the maximum values of P_u will be limited by:

$$\frac{P_u}{f_c' t^2} \leqq \frac{P_b}{f_c' t^2} = 0.36 \left(\frac{90{,}000}{90{,}000 + f_y}\right) \left(1 + 0.67 \frac{D_s}{t}\right) \quad \ldots\ldots (5)$$

For section controlled by compression, the equation can be obtained as follows:

$$\frac{P_u}{f_c' t^2} = \frac{0.85 p_t m}{\dfrac{3e'}{D_s} + 1} + \frac{1}{\dfrac{12 t e'}{(t + 0.67 D_s)^2} + 1.18} \quad \ldots\ldots (6)$$

Circular columns with round cores

For a circular section with round core (Fig. 3), the ACI-ASCE report suggests that the equivalent depth of the concrete section is taken to be $0.8D$, but does not specify explicitly the equivalent width. Since the area of a circle is $0.785D^2$, it is logical to surmise that a width equivalent to D is a close approximation so that the area t^2 for a square section is replaced by $0.8D^2$ for a circular section. The ACI-ASCE report also recommends different assumptions for sections controlled by tension and compression. In the case of tension failure, it is assumed by the report that 0.4 of the total steel area is effective on each side of the section and that the effective $(d - d')$ equals to $0.75D_s$. These assumptions enter into the equations of equilibrium and finally affect Eq. (7). In the case of compression failure, the ACI-ASCE report carried the same assumptions for square columns with round core—that half of the total steel area is effective on each side of the section and that the effective $(d - d')$ equals to $0.67D_s$. These latter assumptions are evident in Eq. (9). This split in assumptions raises the question whether the effective depth of the section for Eq. (8) should be taken as $\frac{1}{2}(0.8D + 0.75D_s)$ or as $\frac{1}{2}(0.8D + 0.67D_s)$. In the charts by Whitney and Cohen,[2] the intersections of tension and compression curves define the limiting loads. Hence, such question does not enter into the problem, and the intersections represent only approximate values of P_u. Since $d = \frac{1}{2}(0.8D + 0.75D_s)$ gives P_u closer to the values indicated in their charts, it is perhaps a better criterion.

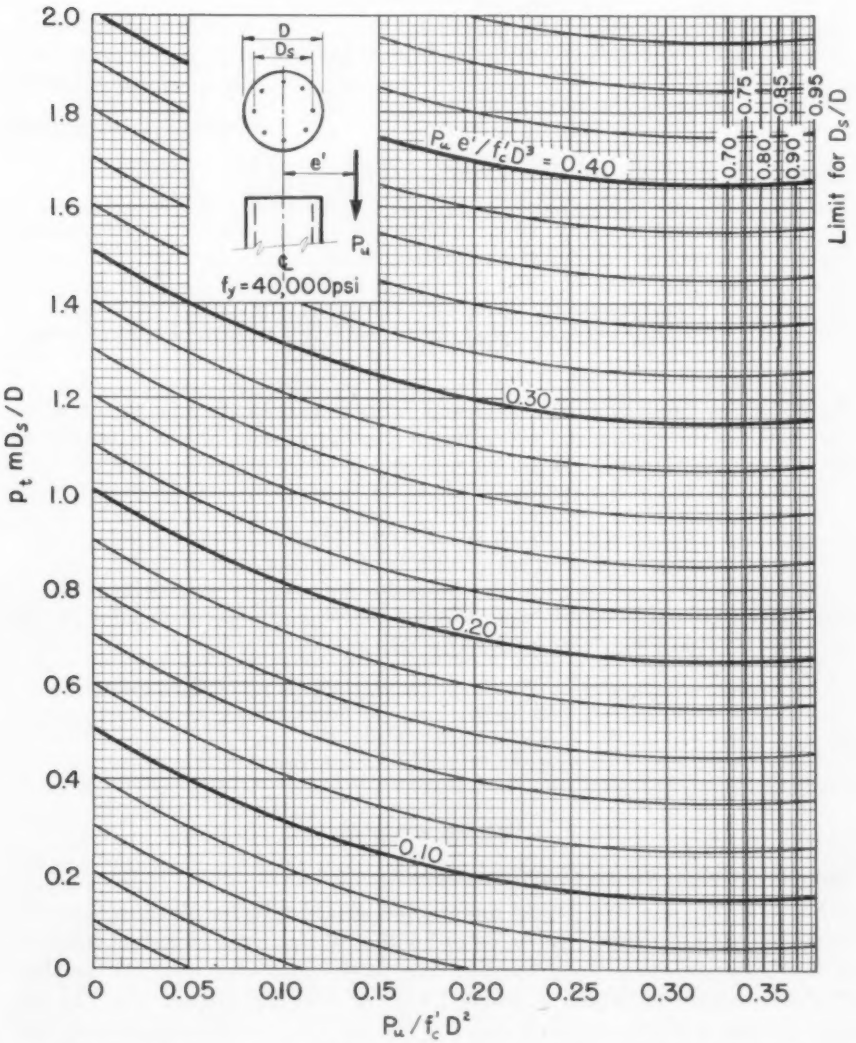

Fig. 3—Circular column with spiral reinforcement controlled by tension*

*A set of nine design charts including Fig. 1, 2, and 3, plus similar charts for f_y = 50,000 psi and 60,000 psi, is available in large size (about 12 x 12 in.) from ACI headquarters at $2.25 per set.

From Appendix C of the ACI-ASCE report, the equations of equilibrium for circular section with round core can be obtained as follows:

$$P_u = 0.85 f_c' A$$

$$P_u e' = 0.85 f_c' A \bar{x} + 0.4 A_{st} f_y (0.75 D_s)$$

where $\bar{x}$ is given in the report as

$$\bar{x} = 0.211 D + 0.293 \left(0.785 D - \frac{2A}{D}\right)$$

Hence, for a section controlled by tension,

$$\frac{P_u e'}{f_c' D^3} = 0.442 \left(\frac{P_u}{f_c' D^2}\right) - 0.69 \left(\frac{P_u}{f_c' D^2}\right)^2 + 0.2 p_t m \frac{D_s}{D} \quad \ldots\ldots (7)$$

By using $d = \frac{1}{2} (0.8 D_s + 0.75 D_s)$ and by taking D as the equivalent width and $0.8D$ as the equivalent depth of the section, the maximum values of P_u for different values of D_s/D will be:

$$\frac{P_u}{f_c' D^2} \leqq \frac{P_b}{f_c' D^2} = 0.36 \left(\frac{90{,}000}{90{,}000 + f_y}\right)\left(0.8 + \frac{0.75 D_s}{D}\right) \quad \ldots\ldots (8)$$

For a section controlled by compression, the equation recommended in the ACI-ASCE report can be expressed in the following form:

$$\frac{P_u}{f_c' D^2} = \frac{0.667 p_t m}{\frac{3e'}{D_s} + 1} + \frac{0.785}{\frac{9.6 D e'}{(0.8 D + 0.67 D_s)^2} + 1.18} \quad \ldots\ldots (9)$$

DESIGN CHARTS

Graphical representation of the equations

Eq. (1), (4), and (7) can be represented graphically by plotting the term involving percentage of steel p_t as ordinate and the term including ultimate load P_u as absicissa, while treating the term containing ultimate moment $P_u e'$ as a parameter which is assigned a set of constant values. Eq. (2), (5), and (8), which set the limits of Eq. (1), (4), and (7) respectively, contain also the variable f_y. With different grades of steel, f_y can be taken as 40,000 psi, 50,000 psi and 60,000 psi. In this paper, design charts for only one grade of steel, i.e. f_y = 40,000 psi, are shown for each type of column section.* (See Fig. 1, 2, and 3.)

*A set of nine design charts including Fig. 1, 2, and 3, plus similar charts for f_y = 50,000 psi and 60,000 psi, is available in large size (about 12 x 12 in.) from ACI headquarters at $2.25 per set.

The limits of the variables in the charts are so chosen that they cover the widest possible range for practical problems. Since the ACI Building Code permits the use of 8 percent of steel reinforcement in rectangular tied columns as well as for square and circular columns with spiral reinforcement, the limit of $p_t m$ has been taken to be 2.0 or greater for all three types of column sections. The limits of t_s/t, D_s/t, and D_s/D also cover most practical cases and can be interpolated very easily.

Illustrative examples

A few numerical examples are given below to illustrate the use of the charts.

1. Given $P_u = 300$ kips, $e' = 15$ in., design a rectangular column with symmetrical reinforcement, using $f_c' = 3000$ psi, $f_y = 40{,}000$ psi, and $m = 40/(3 \times 0.85) = 15.7$.

Try a section $bt = 16$ x 20 in. and assume $d' = 2.5$ in. Then $t_s/t = 15/20 = 0.75$. Enter Fig. 1 with $P_u/f_c'bt = 300/(3 \times 16 \times 20) = 0.313$ and proceed vertically until it reaches $P_u e'/f_c'bt^2 = 300 \times 15/[3 \times 16 \times (20)^2] = 0.235$. Read the value of $p_t m t_s/t = 0.33$ from the chart. Hence, $p_t = 0.33/(15.7 \times 0.75) = 0.028$. Finally, $A_{st} = 0.028 \times 16 \times 20 = 8.96$ sq in.

Note that if the percentage of reinforcement is specified and the size of column is sought to suit the requirement, the solution by chart will not be direct, but the result can be obtained by trial and error.

2. Given $P_u = 600$ kips, $e' = 20$ in., design a rectangular column of minimum size with symmetrical reinforcement, using $f_c' = 3750$ psi, $f_y = 40{,}000$ psi, and $m = 40/(3.75 \times 0.85) = 12.5$.

Enter the limit of t_s/t in Fig. 1 by estimating $t_s/t = 0.70$. From the chart, the maximum value of $P_u/f_c'bt = 0.425$. Thus, the minimum $bt = 600/(3.75 \times 0.425) = 376$ sq in. Try $bt = 18 \times 22 = 396$ sq in. Assume $d' = 3.5$ in. for two layers of bars on each side and $t_s/t = 15/22 = 0.68$. Hence, $P_u/f_c'bt = 600/(3.75 \times 18 \times 22) = 0.403$ which is within the limiting value in the chart. With $P_u e'/f_c'bt^2 = 600 \times 20/[3.75 \times 18 \times (22)^2] = 0.367$, $p_t m t_s/t = 0.600$ is read from the chart. Then $p_t = 0.60/(12.5 \times 0.68) = 0.0705$ and $A_{st} = 27.9$ sq in.

If the column is limited to $t = 18$ in. but not limited in the direction of b, e.g. for architectural reasons, then $b = 22$ in. can be tried. $t_s/t = 11/18 = 0.61$. In this case, $P_u/f_c'bt = 0.403$ still holds true, but $P_u e'/f_c'bt^2 = 600 \times 20/[3.75 \times 22 \times (18)^2] = 0.448$. From Fig. 1, $p_t m t_s/t = 0.80$. Then $p_t = 0.80/(12.5 \times 0.61) = 0.105$. This exceeds the 8 percent of steel specified by the ACI Building Code. Hence, the column size must be increased in the direction of b and the corresponding percentage of steel be recomputed.

3. Given $P_u = 200$ kips acting on a square column 16 x 16 in. with nine #9 bars arranged to form a round core, review this column by finding the maximum eccentricity e' for which the column was designed, using $f_c' = 3000$ psi, $f_y = 40{,}000$ psi and $m = 40/(3 \times 0.85) = 15.7$.

Assume $2\frac{1}{2}$ in. to be the minimum distance from the outer fiber of concrete to center of vertical reinforcing bars. $D_s = 11$ in. and $D_s/t = 0.687$. Compute $P_u/f_c't^2 = 200/[3 \times (16)^2] = 0.26$. $p_t = 9/(16 \times 16) = 0.0351$ and $p_t m D_s/t = 0.0351 \times 15.7 \times 0.687 = 0.379$. Enter Fig. 2 with the computed ordinate and abscissa, and read the value of $P_u e'/f_c't^3 = 0.195$. Hence, $e'/t = 0.195/0.26 = 0.75$, or $e' = 0.75 \times 16 = 12$ in.

Again note that if e' is given and P_u is to be found, the solution by chart will not be convenient, and trial and error method must be used.

4. Given $P_u = 400$ kips, $e' = 20$ in., design a circular column with round core of minimum size, using $f_c' = 3750$ psi, $f_y = 40{,}000$ psi, and $m = 40/(3.75 \times 0.85) = 12.5$

Enter the limit of D_s/D in Fig. 3 by estimating $D_s/D = 0.70$. The maximum value of $P_u/f_c'D^2 = 0.333$ in the chart. Then, the minimum $D^2 = 400/(3.75 \times 0.333) = 320$ sq in. Try D = 18 in. and $D_s = 13$ in. $D_s/D = 0.72$. Then $P_u/f_c'D^2 = 400/[3.75 \times (18)^2] = 0.329$

which is within the limiting value, and $P_u e'/f_c' D^3 = 0.366$. Read the value of $p_t m D_s/D = 1.480$ from the chart. Hence, $p_t = 1.480/(12.5 \times 0.72) = 0.164$. Since 8 percent is the maximum permitted in the ACI Building Code, the size of the column must be increased.

Next try $D = 22$ in. and $D_s = 17$ in. $D_s/D = 0.775$. By repeating the same procedure, $P_u/f_c' D^2 = 0.221$ and $P_u e'/f_c' D^3 = 0.201$. Then, $p_t m D_s/D = 0.68$ from the chart and $p_t = 0.070$ which is below the 8 percent limit. $A_{st} = 26.6$ sq in.

ACKNOWLEDGMENT

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Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1958, for publication in the June 1958 JOURNAL

A set of nine design charts including Fig. 1, 2, and 3, plus similar charts for $f_y = 50{,}000$ psi and 60,000 psi, is available in large size (about 12 x 12 in.) from ACI headquarters at $2.25 per set.

Title No. 54-27

Long-Time Study of Cement Performance in Concrete

Chapter 10—Progress Report on Strength and Elastic Properties of Concrete*

By PAUL KLIEGER†

SYNOPSIS

This report presents strength data through 3 years for laboratory mortar and concrete specimens made using cements of the Long-Time Study. It also presents information on the elastic properties of the concretes as determined by both static and dynamic means, and pulse velocities as determined by the soniscope.

INTRODUCTION

The Long-Time Study of Cement Performance in Concrete embraces numerous laboratory and field studies designed to provide first, a comparison of the performance of cement in structures, and second, the relation between differences in performance and differences in the definable properties of the cements or concretes. This particular study was undertaken to provide data on the strength-producing properties of these Long-Time Study cements when used in concrete.

Details concerning the various field and laboratory studies, together with detailed information on the manufacture of the cements, appear in several ACI and ASTM publications which have been compiled as bulletins of the research laboratories of the Portland Cement Association.[1-6]

TEST PROGRAM

Scope of tests

The Long-Time Study cements were used in making concrete beams for strength tests to be made after different periods of moist curing up to 50 years. Cylinders were made for test at 28 days. Each cement was used in mixes of equal slump at two different water-cement ratios (cement contents of 4½ and 6 bags per cu yd). In addition to strength tests, determinations were made of the elastic constants by the dynamic method, Young's modulus by the static method, and pulse velocity by means of a soniscope.

Mortar briquets, cubes, and prisms were made for strength tests up to 50 years.

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TABLE 1—COMPARISON OF IGNITION LOSS AND NORMAL CONSISTENCY FOR ORIGINAL CEMENTS AND AFTER 10-YEAR STORAGE

Cement No.	Cement type	Ignition loss, percent by weight		Normal consistency of neat cement paste, percent by weight	
		1941 ASTM C 114-40	1951 ASTM C 114-47	1941 ASTM C 77-40	1951 ASTM C 187-49
		TYPE I CEMENTS			
11	I	1.0	1.2	23.0	23.0
11T	IT*	1.2	1.0	23.5	25.0
12	I	0.6	0.8	26.0	23.0
12T	IT*	0.7	0.8	23.5	23.0
13	I	1.8	1.7	23.5	23.5
14	I	0.9	1.6	25.0	23.0
15	I	0.9	0.9	24.5	26.5
16	I	1.1	0.7	23.0	23.0
16T	IT*	1.3	0.9	23.0	23.5
17	I	0.9	0.6	23.0	23.0
18	I	1.0	1.1	24.0	23.5
18T	IT*	0.9	1.0	25.0	26.0
19A	I	—	0.8	—	22.0
19B	I	—	0.9	—	22.5
19C	I	—	1.3	—	25.0
		TYPE II CEMENTS			
21	II	0.7	0.8	23.0	24.0
21T	IIT*	0.6	0.7	23.5	23.5
22	II	0.6	0.7	22.5	23.0
23	II	0.7	0.6	24.0	24.0
24	II	1.4	2.0	24.0	23.0
25	II	0.6	0.4	23.0	23.5
		TYPE III CEMENTS			
31	III	1.5	1.5	27.5	29.0
33	III	1.5	1.4	26.5	26.0
33T	IIIT*	1.4	1.4	26.0	27.0
		TYPE IV CEMENTS			
41	IV	1.3	1.3	22.5	22.5
42	IV	0.9	1.1	24.0	24.0
43	IV	0.7	0.8	25.0	24.0
43A	IV	0.8	1.0	25.0	24.5
		TYPE V CEMENT			
51	V	0.8	0.7	24.0	24.0

*"Treated" cements. Comparable to present-day air-entraining cements.

Materials

Cements—The cements used were 26 of the original 27 Long-Time Study cements,[1] the exception being Cement 34 (supply exhausted). In addition, three cements prepared at a later date for another field project were added to the study making a total of 29 cements.

The original cements were classified as to ASTM type as shown in Table 1. The "treated" cements were obtained by intergrinding flake Vinsol resin. At the time these cements were prepared there were no specifications for air-entraining cements. The three cements added at a later date, 19A, 19B, and 19C, while classed as Type I are further described as follows:

Cement 19A—A so-called old fashioned cement which has a composition and fineness purported to be similar to cements produced in 1924.

Cement 19B—A "modern" coarse-ground cement representative in composition of 1949 Type I cement but ground to approximately the same fineness (1330 Wagner) as 19A.

Cement 19C—A "modern" regular grind cement representative of 1949 specifications for Type I cement, both in composition and fineness.

Chemical and physical properties of the original cements are shown in a previous publication.[1] Chemical and physical properties of Cements 19A, 19B, and 19C are shown in Table 2 of this report.

Since their preparation in 1940 and 1941, the original 26 cements have been stored in sealed metal drums. Cements 19A, 19B, and 19C were manufactured in 1949 and have been stored in sealed metal drums since that time. All of

TABLE 2—CHEMICAL AND PHYSICAL PROPERTIES OF CEMENTS 19A, 19B, AND 19C

	Cement		
	19A	19B	19C
Results of chemical analysis, percent			
SiO_2	22.1	21.6	20.3
Al_2O_3	5.7	5.5	5.8
Fe_2O_3	3.1	2.8	2.9
CaO	62.5	63.9	63.4
MgO	3.4	3.1	3.4
SO_3	1.7	1.7	2.2
Na_2O	0.25	0.24	0.24
K_2O	0.66	0.52	0.56
Loss on ignition	0.7	0.6	1.3
Calculated potential compound composition,* percent			
C_3S	36.8	48.6	52.0
C_2S	35.5	25.2	19.1
C_3A	9.8	9.9	10.3
C_4AF	9.5	8.4	9.0
$CaSO_4$	2.9	2.8	3.7
Free CaO	0.5	0.3	0.6
Results of physical tests			
Specific surface, sq cm per g			
Wagner	1320	1330	1630
Blaine	2500	2470	3200
Passing No. 325 sieve, percent	71.1	72.8	86.1
Specific gravity (kerosene)	3.184	3.172	3.136
Normal consistency, percent	22.0	22.5	25.0
Time of setting: Vicat, hr:min			
Initial	3:30	3:40	3:00
Final	6:40	6:30	6:20
Autoclave expansion, percent	0.23	0.14	0.12

*Corrected for free lime.

the cements appeared in excellent condition after storage, with the exception of LTS 14 and 24. These two cements contained a considerable amount of lumps. Prior to use, these two cements were therefore screened through a No. 30 sieve. Determinations of loss on ignition and normal consistency of neat cement paste are shown in Table 1 together with similar data determined in 1941. There were no outstanding differences resulting from the 10-year storage.

Aggregates—The fine and coarse aggregates were partly siliceous and partly calcareous natural sand and gravel from Elgin, Illinois. These aggregates were similar to the Plainfield, Ill., sand and gravel used in test piles for Cape Cod and Saugerties, N. Y., and in specimens at Naperville, Ill.

The gradation of the Elgin aggregates was as follows:

Elgin sand		Elgin gravel	
Sieve No.	Percent passing	Sieve size or No.	Percent passing
4	100	1½-in.	100
8	82	¾-in.	50
16	67	⅜-in.	25
30	43	No. 4	0
50	13		
100	5		

Mortars and concretes

The cements were used in the preparation of mortar briquets (ASTM C 190-49), 2-in. plastic mortar cubes (ASTM C 109-49), and 1:2.75 plastic mortar prisms (2 x 2 x 11¼ in.). Companion specimens were prepared for both water storage at 73 F (ASTM) and moist air storage at 73 F. Mortar tests were scheduled at 1, 3, 7, and 28 days; 3 months; 1, 3, 10, 25, and 50 years.

All of the cements were used in concrete mixes of two different net water-cement ratios (cement contents maintained at 6 and 4½ bags per cu yd ± 0.05 bag). The slump of both mixes was 2 to 3 in. For the non-air-entraining cements, sand content by absolute volume of total aggregate was 37 percent for the 6-bag mixes and 40 percent for the 4½-bag mixes. For concretes made with "treated" cements, the sand contents were reduced in amounts proportional to the amount of entrained air.

Three of the cements (LTS 12, 23, and 43A) were used in concretes having a cement content of 3 bags per cu yd. Neutralized Vinsol resin was added at the mixer to entrain 4½ ± ½ percent air. Concrete slump was 2 to 3 in.

Concrete tests were scheduled at 1, 7, and 28 days; 3 months; 1, 3, 10, 25, and 50 years.

Test procedures

Mortar—The mortar briquets were tested in accordance with ASTM C 190-49, the cubes in accordance with ASTM C 109-49. The mortar prisms

were tested in flexure with the load applied at the center of a 6-in. span. Two flexural breaks were obtained for each prism. The prism ends were tested in compression as 2-in. modified cubes.

Concrete—Before strength testing of the concrete specimens, various determinations of other physical characteristics were made, such as:

Sonic apparatus—Young's modulus, shear modulus, and Poisson's ratio.
Soniscope—Velocity of a compressional wave through the concrete.
Compressometer—Static Young's modulus for the concrete cylinders by determining the stress-strain curve in compression.
Deflectometer—Static Young's modulus for the concrete beams by determining the load-deflection curve. Calculation was made by the following formula which was derived in a manner comparable to that used for center loading:[7]

$$E = \frac{P}{f}\left[0.96 + \frac{1}{4}\left(\frac{4}{5} + \frac{u}{2}\right)\right]$$

where

E = static Young's modulus of elasticity, psi,
P = total load in lb on 6 x 6-in. beam loaded at the third points of an 18-in. span. (ASTM C 78-49),
f = deflection at center of span, in., and
u = Poisson's ratio as determined for the particular specimen by sonic test.

The term in brackets is a correction factor which takes into account the effect of shear on the measured deflection.

Compressive strength tests of cylinders were made in accordance with ASTM C 39-49. A proprietary capping compound containing sulfur was used to cap the top end. The bottom of the cylinder was cast against a machined steel plate. Two flexural breaks were obtained for each beam specimen. The beam ends were tested in compression as 6-in. modified cubes in accordance with ASTM C 116-49.

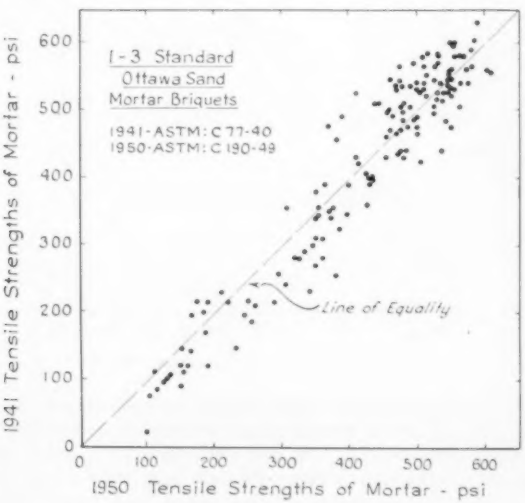

Fig. 1—Comparison of tensile strength of mortars made with LTS cements in 1941 and 1950. Test ages: 1, 3, 7, and 28 days, 3 months, and 1 year

DISCUSSION OF RESULTS

Mortar tests

The results over a 3-year period of tensile, compressive, and flexural tests of mortars made with these cements are shown in Tables 3, 4, 5, and 6. Fig. 1 compares mortar tensile strengths obtained in 1941 with those obtained in 1950, the strengths in general showing fair agreement. For the mortars made in 1950, the strengths at an early age were generally higher and the later-age strengths slightly lower than for the mortars made in 1941. This may be attributed to the difference in fabrication and curing temperature in

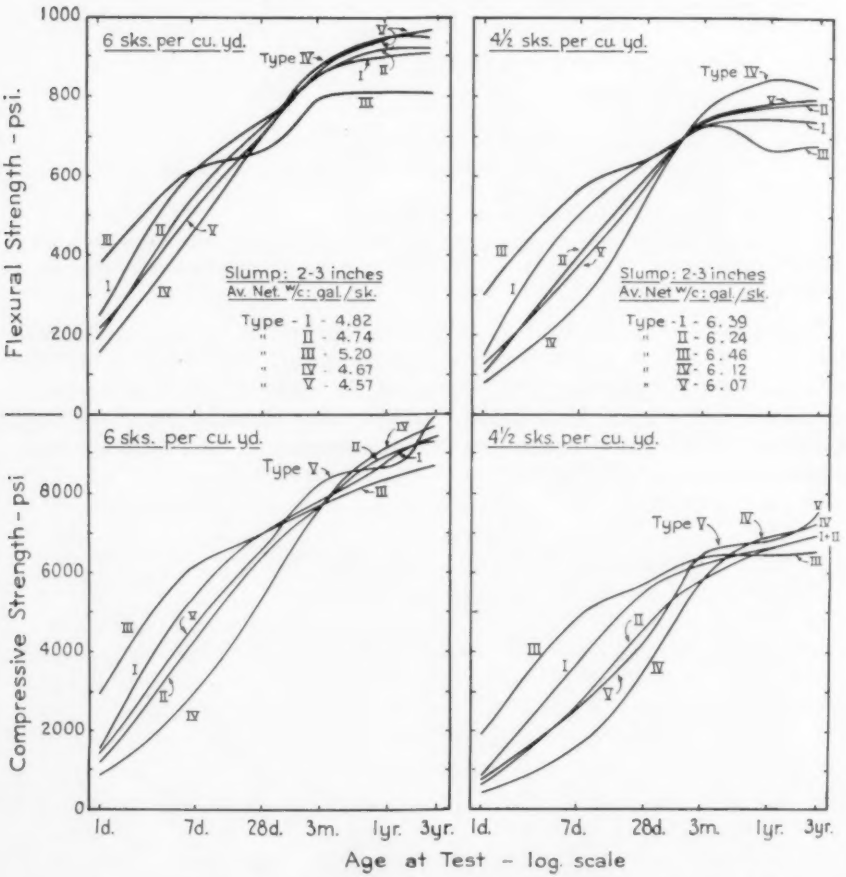
Flexural Strength - psi.
Compressive Strength - psi
6 sks. per cu. yd.
4½ sks. per cu. yd.
Slump: 2-3 inches
Av. Net w/c: gal./sk.
Type - I - 4.82
" II - 4.74
" III - 5.20
" IV - 4.67
" V - 4.57
Type - I - 6.39
" II - 6.24
" III - 6.46
" IV - 6.12
" V - 6.07
1d. 7d. 28d. 3m. 1yr. 3yr.
Age at Test - log. scale

Fig. 2—Age-strength relationships for concretes made with different types of cement. Results are the average of all the non-air-entraining cements in one type. (For the Type I cements 19A, 19B, and 19C were excluded.) Specimens: 6 x 6 x 30-in. beams cured continuously moist. Flexure test: third-point loading on an 18-in. span, two breaks per beam. Compression test: beam ends as 6-in. modified cubes

TABLE 3—TENSILE STRENGTH OF 1:3 STANDARD OTTAWA SAND BRIQUETS*

Cement No.	Tensile strength, 1:3 mortar briquets, psi													
	Water storage (ASTM)							Moist air storage						
	1 day	3 days	7 days	28 days	3 months	1 year	3 years	1 day	3 days	7 days	28 days	3 months	1 year	3 years
Type I Cements														
11	210	385	465	525	470	455	430	210	375	475	555	560	480	460
11T†	165	310	380	440	410	370	350	170	320	410	465	400	400	390
12	150	380	470	555	550	490	485	150	400	475	585	575	530	500
12T†	125	340	480	525	545	500	470	115	355	490	550	510	500	485
13	190	375	455	550	565	550	485	185	380	470	535	535	520	520
14	230	370	460	545	540	495	475	220	360	465	565	590	505	515
15	245	480	525	555	535	510	445	240	495	550	610	575	560	520
16	250	400	475	505	515	490	465	245	430	490	575	535	535	520
16T†	185	350	410	500	460	445	445	205	365	445	475	425	465	480
17	175	435	480	575	530	510	465	180	440	490	530	555	510	495
18	190	425	485	550	550	505	465	170	390	480	535	570	510	460
18T†	100	320	430	500	480	470	395	100	365	405	515	505	450	430
19A	150	310	395	495	515	485	445	145	305	405	525	525	525	450
19B	155	340	445	505	495	480	455	155	335	455	560	565	480	460
19C	250	475	545	555	565	505	485	250	490	555	585	590	525	525
Type II Cements														
21	110	325	435	525	565	530	540	110	325	430	545	585	575	575
21T†	115	305	375	500	545	495	485	105	290	405	535	550	530	535
22	150	325	420	550	545	515	485	140	305	460	555	585	545	515
23	150	355	480	580	555	510	510	150	350	470	560	555	545	535
24	190	355	415	525	530	490	450	190	390	465	520	555	530	460
25	165	295	395	540	550	530	475	170	295	400	515	520	545	500
Type III Cements														
31	350	545	605	590	555	510	490	340	550	655	610	605	530	530
33	365	500	570	555	550	460	465	385	525	580	570	570	495	480
33T†	350	455	470	475	475	390	335	330	470	490	500	470	450	425
Type IV Cements														
41	155	290	360	485	530	475	465	155	285	370	500	545	505	460
42	135	260	350	505	610	515	550	125	280	340	535	620	530	530
43	105	255	360	475	565	510	510	105	250	355	525	555	515	490
43A	130	220	335	535	575	510	495	120	230	335	515	595	545	595
Type V Cement														
51	160	345	430	545	585	580	515	150	345	405	575	595	590	605

*Each value is the average of three specimens. Made and tested in accordance with ASTM C 190-49. Duplicate specimens prepared for curing in moist air at 73.4 ± 3F and 100 percent relative humidity.

†"Treated" cements. Comparable to present-day air-entraining cements.

TABLE 4—COMPRESSIVE STRENGTH OF 1:2.75 GRADED OTTAWA SAND PLASTIC MORTAR CUBES (2-IN.)*

Cement No.	Compressive strength, 1:2.75 mortar cubes, psi													
	Water storage (ASTM)							Moist air storage						
	1 day	3 days	7 days	28 days	3 months	1 year	3 years	1 day	3 days	7 days	28 days	3 months	1 year	3 years
Type I Cements														
11	650	1850	3340	5220	5540	5220	5460	670	1890	3670	5430	5730	5630	5630
11T†	520	1650	2950	4160	4270	4320	4270	520	1600	3040	4150	4470	4430	4420
12	490	1850	3280	5260	6080	5950	5860	490	1840	3350	5520	6360	6100	5940
12T†	350	1580	2910	4800	5450	5250	5330	340	1540	2910	4830	5770	5370	5400
13	720	1560	2350	4140	5330	5660	5810	720	1570	2410	4210	5530	5950	5880
14	690	1730	2880	4330	4950	5140	5210	720	1670	2870	4350	4990	5290	5540
15	1010	3060	4750	5870	5750	5530	5090	1000	3110	4430	6130	5940	5890	5710
16	840	2170	3560	5100	5610	5580	5790	810	2120	3590	5170	5680	5920	5910
16T†	560	1420	2380	3350	3990	4030	4050	560	1450	2360	3560	4160	4100	4230
17	630	2160	3530	5610	5900	5700	5420	620	2110	3630	6000	6100	6100	5790
18	580	1920	3520	5480	6040	5990	5580	610	1900	3330	5530	6130	6050	6000
18T†	290	1730	2990	4620	4920	4650	4910	280	1740	2960	4790	5210	5010	4880
19A	360	1010	1610	2950	4050	4210	4390	350	1100	1610	2870	4170	4690	4680
19B	430	1180	2180	3700	4340	4370	4510	440	1180	2190	3820	4560	4600	4680
19C	750	2140	3460	5050	5260	5380	4780	760	2170	3580	5160	5430	5520	5680
Type II Cements														
21	380	1380	2450	4330	5940	6670	6810	380	1350	2410	4450	6350	6850	7140
21T†	320	1200	1980	3750	5470	5890	5860	310	1240	2150	4020	5620	5630	5920
22	410	1200	2340	5000	5950	5960	6230	410	1200	2330	4980	6250	6290	6310
23	620	1660	2580	4580	5420	5810	5760	600	1650	2590	4730	6130	6480	6430
24	630	1530	2270	3670	4600	4790	4970	620	1480	2320	3760	4580	4880	5010
25	550	1160	1770	3790	5280	5640	5900	540	1160	1890	3910	5120	5690	6150
Type III Cements														
31	1820	4000	5630	7010	7150	6900	6550	1820	4310	6180	7640	7770	7540	7080
33	2010	4180	5910	6340	6460	6210	5850	2030	4220	5870	6700	6810	6620	6240
33T†	1560	3330	4480	5190	5340	4900	4320	1560	3460	4790	5500	5340	5220	4920
Type IV Cements														
41	400	950	1340	3230	5070	5910	5880	400	1010	1390	3350	5480	5890	6180
42	390	940	1290	3070	6110	7110	7350	380	920	1250	3120	6270	7500	7680
43	260	860	1480	3620	5440	5810	5650	270	910	1730	3680	5420	5710	5680
43A	410	890	1320	3490	5990	6650	6920	400	840	1320	3690	6390	7060	7190
Type V Cement														
51	620	1500	2180	3870	6090	6850	7190	620	1470	2120	3840	6150	7300	7150

*Each value is the average of three specimens. Made and tested in accordance with ASTM C 109-49. Duplicate specimens prepared for curing in moist air at 73.4 ± 3 F and 100 percent relative humidity.

†"Treated" cements. Comparable to present-day air-entraining cements.

TABLE 5—FLEXURAL STRENGTH OF 1:2.75 GRADED OTTAWA SAND PLASTIC MORTAR PRISMS*

Cement No.	Flexural strength, 1:2.75 mortar prisms, psi										
	Water storage					Moist air storage					
	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years
Type I Cements											
11	610	790	750	740	755	195	655	815	765	755	745
11T†	565	660	655	615	635	160	600	680	670	625	625
12	665	840	815	815	795	140	695	880	885	845	825
12T†	580	735	785	725	735	95	630	795	800	730	730
13	545	755	815	830	830	200	585	790	885	885	890
14	560	755	785	765	805	180	575	700	750	775	800
15	765	895	795	785	775	290	795	900	880	825	810
16	675	810	820	800	855	230	685	815	855	845	850
16T†	585	715	715	740	765	205	605	735	750	760	745
17	660	835	800	755	755	155	665	825	840	810	825
18	655	810	830	795	795	165	660	830	855	790	800
18T†	590	725	725	700	725	95	610	775	755	735	695
19A	385	580	680	715	730	95	425	595	740	735	690
19B	525	700	765	745	765	130	550	745	780	760	755
19C	655	810	820	760	780	190	685	825	840	795	760
Type II Cements											
21	535	785	910	915	890	105	555	785	915	950	935
21T†	475	665	810	835	855	95	485	730	855	860	850
22	520	800	850	800	840	125	565	830	925	855	880
23	570	820	865	865	900	145	610	860	870	870	895
24	550	685	765	775	805	170	570	710	785	755	785
25	480	775	895	860	920	175	510	760	865	855	845
Type III Cements											
31	895	945	890	885	900	405	845	925	930	885	850
33	890	835	825	810	825	495	870	825	845	805	805
33T†	740	775	715	750	750	430	730	730	760	755	715
Type IV Cements											
41	360	680	840	850	820	105	380	670	820	820	805
42	325	645	940	930	945	105	340	700	975	935	965
43	390	725	845	830	795	85	405	745	890	820	770
43A	375	725	915	900	920	115	390	740	945	890	925
Type V Cement											
51	505	790	945	975	990	165	490	785	1005	970	1055

*Each value is the average of two tests on each of three specimens, total of six tests. Specimens are 2 x 2 x 11¼-in. prisms tested with the load applied at the center of a 6-in. span. The specimens were prepared in accordance with ASTM C 109-49, except as relates to specimen size, molding, and testing. Duplicate specimens prepared for curing in moist air at 73.4 ± 3 F and 100 percent relative humidity.

†"Treated" cements. Comparable to present-day air-entraining cements.

the ASTM methods current for these 2 years. In 1941, this temperature was 70 F, while in 1950 the temperature was 73 F. This differential of 3 F would probably be sufficient to influence the strength gain characteristics of the mortars in this manner.

Retrogressions in mortar strengths apparent after 28 days of either water or moist air storage are normal occurrences in tests of this type.

Concrete tests

Table 7 shows the cement contents, net water-cement ratios, sand percentages, slumps, and air contents for both the 4½ and 6 bags per cu yd mixes. Similar data for the 3 bags per cu yd mixes made with three cements are shown in Table 8.

TABLE 6—COMPRESSIVE STRENGTH OF 1:2.75 GRADED OTTAWA SAND PLASTIC MORTAR MODIFIED CUBES (2-IN.)*

Cement No.	Compressive strength, 1:2.75 mortar modified cubes, psi									
	Water storage					Moist air storage				
	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years
Type I Cements											
11	3470	4870	5320	5200	4920	700	3970	5050	5180	5440	5720
11T†	3000	4100	4150	4120	4220	560	3220	4220	4370	4380	4320
12	3670	5630	5850	5990	5980	520	3960	5800	6180	6350	6190
12T†	2960	4410	5210	4960	5330	360	3200	4880	5500	5400	5270
13	2600	4520	5210	5260	5580	730	2870	4660	5700	5920	6040
14	2920	4060	4580	4630	5040	660	3010	4090	4560	5020	5000
15	4700	6070	5380	5450	5260	1120	4810	6070	6040	5700	5780
16	3680	5160	5530	5530	5740	840	4190	5230	5610	5810	6150
16T†	2820	4050	4250	4430	5020	690	3110	4260	4340	5010	4950
17	3450	5290	5520	5320	5500	560	3780	5600	5890	5730	6000
18	3460	5230	5810	5720	5960	600	3820	5590	6100	5860	5910
18T†	3210	4540	4870	4860	5000	310	3170	4680	4890	5070	5010
19A	1520	2580	3420	3710	4310	280	1660	2910	3860	4430	4520
19B	2340	3630	4240	4150	4380	460	2690	3950	4380	4740	4810
19C	3280	4520	4800	4700	4880	690	3500	4830	5090	5150	5170
Type II Cements											
21	2480	4560	5920	6240	6700	370	2800	4600	5730	6990	6990
21T†	2270	3980	5180	5720	6200	350	2260	4080	5400	5950	6140
22	2530	4770	5680	5440	5710	440	2820	5100	5840	6470	6530
23	2620	4700	5480	5850	5880	580	3030	5200	5650	6370	6130
24	2590	3710	4220	4410	4850	600	2780	3910	4390	4740	4780
25	2100	3940	5090	5350	5820	600	2290	4060	4820	5350	5550
Type III Cements											
31	5900	7300	7420	6840	6880	1760	5780	7150	7190	7060	7130
33	5620	6100	6200	5850	5910	2030	5580	5840	6060	6190	6110
33T†	4320	5080	5260	5260	5370	1740	4250	5120	5120	5300	5320
Type IV Cements											
41	1540	3330	4830	5380	5690	360	1610	3590	4900	5620	5670
42	1270	3100	5660	6200	6610	350	1360	3420	6050	7070	6930
43	1680	3700	5080	5460	5580	290	1760	3730	5340	5520	5540
43A	1520	3810	6040	6710	6730	390	1630	3850	6070	6660	6880
Type V Cement											
51	2400	4280	6320	6880	7060	630	2320	4310	6340	7420	7220

*Each value is the average of tests on both ends of three 2 x 2 x 11¼-in. prisms prepared in accordance with ASTM C 109-49, except as relates to specimen size, molding, and testing. Duplicate specimens prepared for curing in moist air at 73.4 ± 3 F and 100 percent relative humidity.

†"Treated" cements. Comparable to present-day air-entraining cements.

Flexural and compressive strengths—Flexural and compressive strengths of these concretes are shown in Tables 9, 10, and 11. Fig. 2 shows the average age-strength relationships through 3 years for each type of cement. Only the data for concretes made with the non-air-entraining cements were included in these averages. In addition, Cements 19A, 19B and 19C were omitted from the group of Type I cements. Fig. 3 shows the age-strength relationships through 3 years for the three cements used in the 3 bags per cu yd air-entrained concretes.

In general, concretes made with the Type I and Type III cements showed higher strengths through 28 days than concretes made with the Type II, IV, and V cements. At 1 year and 3 years, however, concretes made with Type

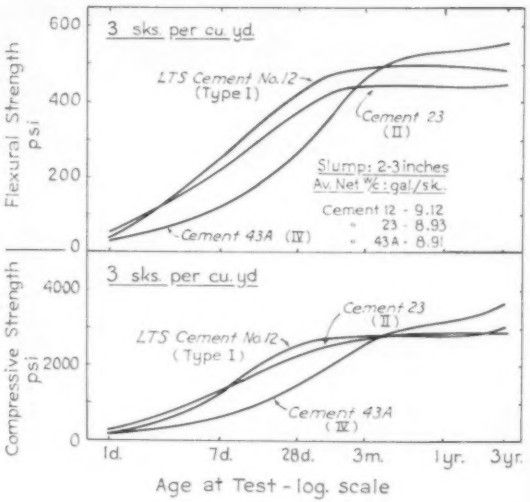

Fig. 3—Age-strength relationships for concretes made with different types of cement, 3 bags per cu yd. Vinsol resin added at mixer to entrain approximately 4½ percent air. Specimens were 6 x 6 x 30-in. beams cured continuously moist. Flexure test: Third-point loading on 18-in. span, two breaks per beam. Compression test: beam ends as 6-in. modified cubes

II, IV, and V cements generally showed higher strengths than concretes made with the Type I and III cements.

A comparison of 28-day strengths of cylinders and modified cubes is shown in Fig. 4.

Elastic properties by dynamic (sonic) method—Tables 12, 13, and 14 show Young's modulus and shear modulus as determined by means of the sonic apparatus and values of Poisson's ratio calculated from these data for the concretes made with all of the LTS cements.

Fig. 5 shows the empirical relationships between Young's modulus (sonic) and both flexural and compressive strengths of the concretes. While the relationships for each cement content are on the whole good, it is apparent that the average relationships differ for the three cement contents. For example, at a compressive strength of 3000 psi, the values for Young's modulus for the 6, 4½, and 3-bag concretes are 4.1, 4.4 and 5.3 $\times$ 10^6 psi, respectively. These average relationships for compressive strength are shown superimposed on one another in Fig. 6. The difference in modulus values at a constant strength appears related to the differences in paste-aggregate ratio. Decreases in this ratio indicate larger amounts of aggregate relative to paste. Dynamic modulus values for

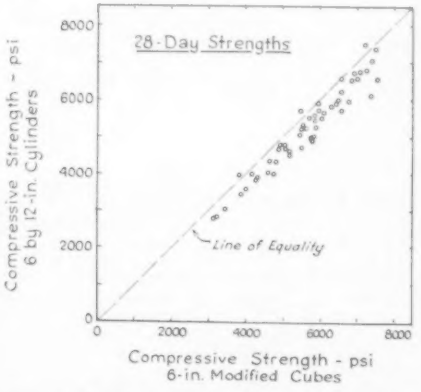

Fig. 4—Relationship between compressive strengths of 6-in. modified cubes and 6 x 12-in. cylinders. Data taken from Tables 9 and 10

TABLE 7—CHARACTERISTICS OF CONCRETES MADE WITH LTS CEMENTS

Cement No.	Nominal: 6 bags per cu yd					Nominal: 4½ bags per cu yd				
	Actual cement content, bags per cu yd	Net W/C, gal. per bag	Sand, percent by absolute volume	Slump, in.	Air content, percent (pressure)	Actual cement content, bags per cu yd	Net W/C, gal. per bag	Sand, percent by absolute volume	Slump, in.	Air content, percent (pressure)
Type I Cements										
11	6.01	4.63	37.0	1.9	1.30	4.50	6.26	40.0	2.5	1.42
11T*	5.95	4.46	32.9	2.4	6.66	4.49	5.28	36.4	2.3	7.58
12	6.00	4.61	37.0	2.3	1.52	4.50	6.15	40.0	2.9	1.73
12T*	6.00	4.55	35.8	2.2	2.77	4.48	5.94	38.7	2.3	3.58
13	6.01	4.80	37.0	2.5	1.12	4.53	6.41	40.0	2.4	1.19
14	6.00	4.63	37.0	2.4	1.63	4.53	6.15	40.0	2.4	1.82
15	6.01	5.07	37.0	2.4	1.24	4.52	6.56	40.0	2.4	1.34
16	6.01	4.63	37.0	1.9	1.38	4.51	6.26	40.0	2.1	1.55
16T*	6.03	4.25	32.7	2.2	5.59	4.47	5.16	35.3	2.4	7.52
17	6.02	5.22	37.0	2.3	1.24	4.50	6.83	40.0	2.2	1.34
18	6.02	4.98	37.0	2.8	1.15	4.54	6.49	40.0	2.6	1.24
18T*	6.01	4.80	35.2	2.5	3.07	4.51	5.77	37.6	1.9	3.93
19A	6.02	5.08	37.0	3.4	1.48	4.52	6.63	40.0	2.7	1.95
19B	6.02	5.09	37.0	2.9	1.28	4.50	6.73	40.0	2.6	1.59
19C	6.01	5.09	37.0	2.5	1.19	4.50	6.55	40.0	2.2	1.45
Type II Cements										
21	6.01	4.88	37.0	2.6	1.14	4.54	6.29	40.0	2.5	1.29
21T*	5.99	4.41	35.1	2.0	3.87	4.50	5.53	37.7	2.1	4.88
22	6.01	4.57	37.0	2.5	1.40	4.51	6.11	40.0	2.7	1.67
23	6.01	4.74	37.0	2.5	1.29	4.53	6.14	40.0	2.3	1.46
24	6.02	4.73	37.0	2.7	1.60	4.53	6.33	40.0	2.8	1.76
25	6.02	4.76	37.0	2.2	1.56	4.55	6.35	40.0	2.4	1.73
Type III Cements										
31	6.01	5.50	37.0	2.9	1.16	4.54	6.71	40.0	3.0	1.23
33	5.99	4.91	37.0	2.3	1.34	4.50	6.20	40.0	2.0	1.61
33T*	5.98	4.55	34.7	2.1	4.60	4.54	5.42	36.5	2.4	5.84
Type IV Cements										
41	6.02	4.53	37.0	2.3	1.85	4.53	6.04	40.0	2.4	1.97
42	6.03	4.79	37.0	2.5	1.22	4.54	6.19	40.0	2.1	1.37
43	6.00	4.72	37.0	2.7	2.06	4.52	6.10	40.0	2.4	2.54
43A	6.00	4.65	37.0	2.7	1.90	4.51	6.14	40.0	2.7	2.21
Type V Cement										
51	6.01	4.57	37.0	2.1	1.29	4.50	60.7	40.0	2.4	1.49

*"Treated" cements. Comparable to present-day air-entraining cements.

TABLE 8—CHARACTERISTICS OF 3 BAGS PER CU YD AIR-ENTRAINED CONCRETES MADE WITH LTS CEMENTS*

Cement No.	Cement type	Actual cement content, bags per cu yd	Net W/C, gal. per bag	Percent sand, by absolute volume	Slump, in.	Air content, percent (pressure)
12	I	3.00	9.12	41.0	2.9	4.83
23	II	3.02	8.93	41.0	2.4	4.30
43A	IV	3.02	8.81	41.0	2.1	4.30

*Neutralised Vinsol resin in solution added at the mixer as air-entraining agent.

TABLE 9—STRENGTH OF CONCRETES MADE WITH LTS CEMENTS, 6 BAGS PER CU YD*

Cement No.	Net *W/C*, gal. per bag	Flexural strength, psi						Compressive strength, psi						
								6-in. modified cubes						6 x 12-in. cylinders
		1 day	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years	28 days
Type I Cements														
11	4.63	240	640	765	840	905	855	1550	5680	7390	7610	9160	9810	7020
11T†	4.46	180	500	600	660	660	710	890	3690	5030	5380	5740	6390	4760
12	4.61	180	640	790	910	925	965	1120	5920	7490	8770	9170	9710	7370
12T†	4.55	160	595	730	850	885	865	860	4930	6920	7740	8720	9330	6700
13	4.80	260	525	690	845	885	915	1520	4320	6280	7560	8280	8620	5850
14	4.63	250	620	725	800	915	890	1490	5020	6460	7160	8730	9280	5920
15	5.07	320	660	755	865	865	870	2230	6080	7180	7820	8530	9290	7490
16	4.63	265	675	755	890	920	935	1820	6040	7230	8080	9540	10160	6780
16T†	4.25	230	510	625	720	780	800	1250	3850	5050	5710	6380	7390	4670
17	5.22	210	600	685	855	870	935	1340	5220	7040	7560	8700	9310	6740
18	4.98	220	585	720	830	915	895	1220	5290	7000	7600	8360	9110	6560
18T†	4.80	140	530	655	760	785	830	720	4750	6310	6500	7450	8080	5800
19A	5.08	145	450	605	745	840	870	770	3090	4810	6600	7350	7930	4320
19B	5.09	195	555	670	810	835	875	1110	4350	6560	7450	7740	8850	5720
19C	5.09	275	605	750	890	920	890	1540	5370	6860	7390	7960	9000	6530
Type II Cements														
21	4.88	140	525	710	845	940	925	860	4110	6510	8160	9640	9580	6000
21T†	4.41	140	490	640	740	870	920	770	3590	5780	6950	8350	9460	4880
22	4.57	170	580	735	910	935	945	1060	5030	7540	8340	9240	9940	6560
23	4.74	215	565	750	870	950	945	1420	4530	6760	7550	9210	9540	5970
24	4.73	240	565	715	780	865	870	1270	4400	5530	6920	8170	8160	5310
25	4.76	220	510	715	850	905	925	1300	3740	5960	7710	8560	9300	5720
Type III Cements														
31	5.50	315	590	655	765	760	790	2110	5730	6540	7350	7800	7970	6560
33	4.91	435	650	665	825	850	815	3690	6580	7360	8140	8840	9340	6100
33T†	4.55	405	635	675	745	815	770	2580	5330	6090	6390	7320	7870	5650
Type IV Cements														
41	4.53	180	495	725	875	980	940	970	3290	5950	7730	9140	9630	5900
42	4.79	160	405	665	890	925	965	800	2460	4880	7720	9470	9640	4660
43	4.72	115	495	750	890	930	905	660	3340	5470	7280	8480	9500	5700
43A	4.65	160	430	650	870	935	965	880	2580	5470	7610	9220	9890	5180
Type V Cement														
51	4.57	215	520	705	860	945	965	1420	4400	6560	8300	8620	9990	6200

*Flexure: 6 x 6 x 30-in. beams. Loaded at third points of 18-in. span. Two breaks per beam.
Compression: 6-in. modified cubes and 6 x 12-in. cylinders.
Each value is the average of six for flexure and modified cubes and three for cylinders. All concretes cured continuously moist at 73.4 ± 3 F.
†"Treated" cements. Comparable to present-day air-entraining cements.

TABLE 10—STRENGTH OF CONCRETES MADE WITH LTS CEMENTS, 4½ BAGS PER CU YD*

Cement No.	Net W/C, gal. per bag	Flexural strength, psi						Compressive strength, psi						
								6-in. modified cubes						6 x 12-in. cylinders
		1 day	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years	28 days
Type I Cements														
11	6.26	135	520	625	710	690	730	780	4210	5870	6390	7020	6910	5250
11T†	5.28	140	445	515	595	585	595	600	3010	4190	4460	4780	5080	3840
12	6.15	100	530	705	785	755	770	580	3800	5710	6650	7010	7380	5510
12T†	5.94	80	495	645	765	790	760	420	3680	6030	6580	7240	7560	5480
13	6.41	165	420	595	705	765	765	890	2740	4730	5760	6400	6750	3990
14	6.15	150	490	630	740	765	745	800	3480	5190	5700	6540	7030	4500
15	6.56	215	580	650	710	710	665	1260	4730	5830	6280	6440	6360	5580
16	6.26	165	535	655	760	770	775	1020	4000	5820	6520	7060	7640	5000
16T†	5.16	155	415	550	635	605	665	800	2940	4150	4480	4770	5490	3970
17	6.83	120	490	650	735	760	720	740	3480	5770	6410	6560	6890	4920
18	6.49	130	510	650	750	740	730	670	3600	5600	6230	6540	6810	5240
18T†	5.77	100	485	585	710	720	720	500	3980	5460	5770	6260	6570	5050
19A	6.63	75	300	470	610	655	700	410	1710	3100	4280	5060	5940	2750
19B	6.73	105	415	580	680	725	715	560	2640	4260	5140	5420	6000	3810
19C	6.55	170	510	665	775	810	765	890	3910	5520	6200	6580	6970	4720
Type II Cements														
21	6.29	75	380	590	745	825	825	540	2600	4630	6080	7410	7630	4320
21T†	5.53	90	420	575	700	810	800	450	2650	4590	5900	6920	7080	4020
22	6.11	95	425	640	760	755	815	560	2800	5150	6000	6620	7170	4600
23	6.14	130	445	615	765	800	765	830	3220	5040	6200	6790	7100	4690
24	6.33	130	440	585	690	735	745	680	2800	4300	5360	5940	6060	3860
25	6.35	105	325	545	730	790	760	620	1980	4000	5630	6580	6820	3580
Type III Cements														
31	6.71	230	540	640	710	645	640	1480	4820	5820	6490	6260	6370	5440
33	6.20	370	585	635	755	695	720	2290	4900	5560	6340	6600	6790	5310
33T†	5.42	340	545	615	655	660	615	1860	4320	4950	5370	5560	5880	4790
Type IV Cements														
41	6.04	95	305	575	745	875	835	510	1760	3860	5920	6940	6830	3410
42	6.19	95	270	500	825	880	860	460	1490	3200	5930	7140	7850	2810
43	6.10	60	325	615	770	770	775	340	1780	3810	5560	6750	7070	3950
43A	6.14	80	265	525	745	845	815	460	1490	3450	5700	6820	7380	3010
Type V Cement														
51	6.07	125	380	595	745	780	795	780	2590	4220	6440	6850	7570	3890

*Flexure: 6 x 6 x 30-in. beams. Loaded at third points of 18-in. span. Two breaks per beam.
Compression: 6-in. modified cubes and 6 x 12-in. cylinders.
Each value is the average of six for flexure and modified cubes and three for cylinders. All concretes cured continuously moist at 73.4 ± 3 F.
†"Treated" cements. Comparable to present-day air-entraining cements.

TABLE 11—STRENGTH OF AIR-ENTRAINED CONCRETES MADE WITH LTS CEMENTS—3 BAGS PER CU YD (VINSOL RESIN ADDED)*

Cement No.	Flexural strength, psi						Compressive strength, psi						
							6-in. modified cubes						6 x 12-in. cylinder
	1 day	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years	28 days
12	35	250	420	490	500	485	170	1240	2550	2820	2780	2990	2360
23	50	225	370	440	440	450	270	1280	2240	2680	2860	2940	2010
43A	30	115	270	460	535	560	160	600	1400	2620	3150	3660	1280

*Flexure: 6 x 6 x 30-in. beams. Loaded at third points of 18-in. span. Two breaks per beam.
Compression: 6-in. modified cubes and 6 x 12-in. cylinders.
Each value is the average of six for flexure and modified cubes and three for cylinders. All concretes cured continuously moist at 73.4 ± 3 F.

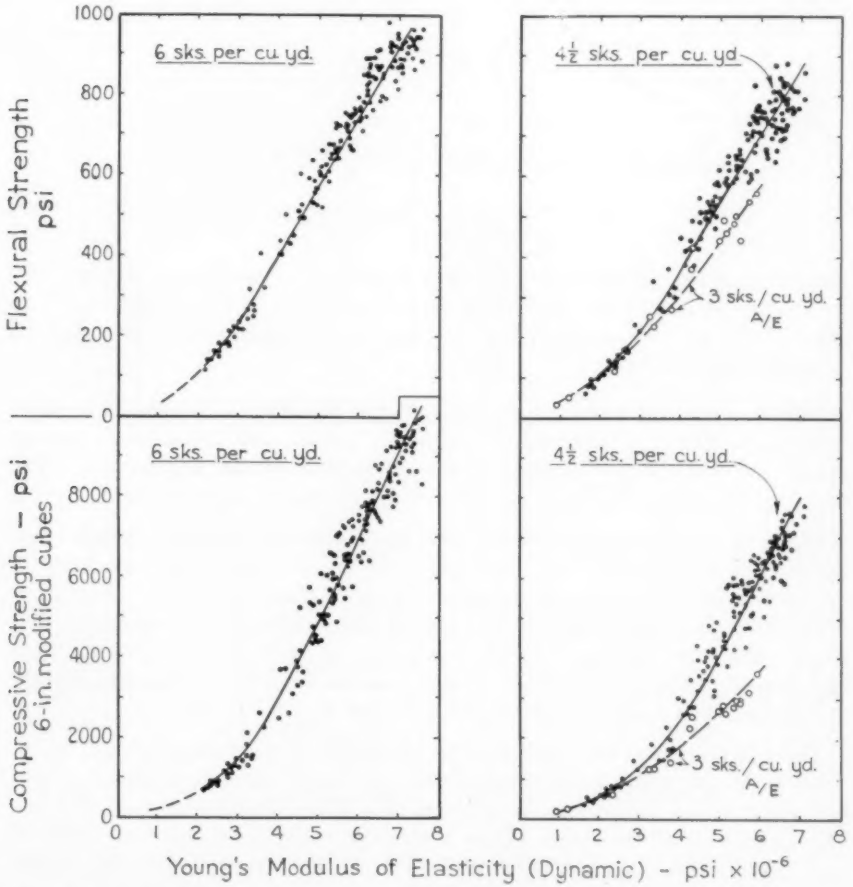
Flexural Strength psi
Compressive Strength - psi 6-in. modified cubes
6 sks. per cu. yd.
4½ sks. per cu. yd.
3 sks./cu. yd. A/E
Young's Modulus of Elasticity (Dynamic) - psi x 10⁻⁶

Fig. 5—Relationships between Young's modulus (dynamic) and strengths of concrete made with different types of cement. Plotted points represent data for individual cements of all five types. Specimens: 6 x 6 x 30-in. beams cured continuously moist. Test ages: 1, 7, and 28 days, 3 months, 1 and 3 years

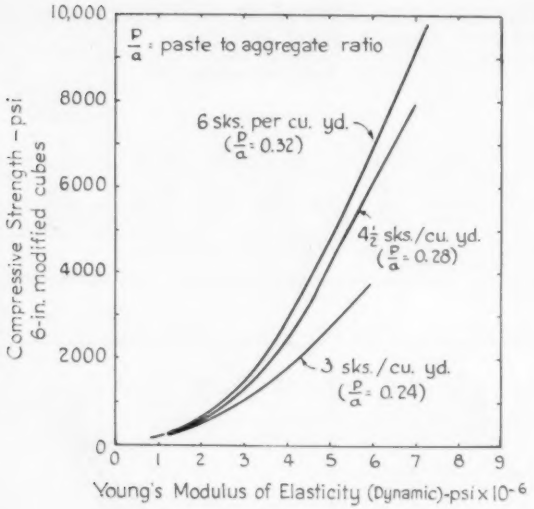

Fig. 6—Effect of paste-aggregate ratio on the relationship between Young's modulus (dynamic) and compressive strength. Average curves taken from Fig. 5

paste in general range up to 2.5 to 3.5 $\times$ 10^6 psi while the dynamic modulus value for the aggregate used in these concretes was approximately 6 $\times$ 10^6 psi. These differences account for changes in the relationships with changes in paste-aggregate ratio.

Elastic properties by static methods—Table 15 shows the values of static modulus of elasticity determined at 28 days, 1 year, and 3 years for concretes made with some of the cements, with each type of cement represented. The dynamic modulus values are also shown for comparative purposes.

Fig. 7 is a graphical comparison of these static and dynamic values. For the 6 x 12-in. cylinders, the static values in all cases are lower than the dynamic values, the differences being smaller at 1 year and 3 years than at 28 days, indicating that creep decreased with increase in age and strength. For the 6 x 6 x 30-in. beams, the static values at 28 days are slightly lower than the dynamic values. At 1 and 3 years some of the static values are slightly higher than the dynamic values and some are slightly lower.

There is no reason to expect perfect agreement between static and dynamic values. In the static tests, the inelastic portion of the measured strain at a particular stress introduces an error in the determination of the modulus. The proportion of the strain which is inelastic varies with age, water-cement ratio, rate of loading, and other variables. In the dynamic test, the fundamental resonant frequency is determined at a very low stress level and consequently this variable is almost completely eliminated. The dynamic testing technique also tends to ascribe more weight to the outer portions of the specimen, which may differ somewhat from the interior.

TABLE 12—ELASTIC PROPERTIES (DYNAMIC) OF CONCRETES MADE WITH LTS CEMENTS, 6 BAGS PER CU YD*

Cement No.	Young's modulus, psi × 10^{-6}						Shear modulus, psi × 10^{-6}						Poisson's ratio					
	1 day	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years
Type I Cements																		
11	3.23	5.29	5.86	6.38	7.18	7.09	1.33	2.15	2.31	2.57	2.82	2.84	.21	.23	.27	.24	.27	.26
11T†	2.52	4.21	5.04	5.40	5.85	6.06	1.08	1.76	2.04	2.13	2.33	2.42	.17	.20	.24	.27	.25	.25
12	2.67	5.40	5.98	6.44	7.06	7.60	1.12	2.12	2.34	2.60	2.81	3.00	.19	.27	.28	.24	.25	.27
12T†	2.43	5.14	5.85	6.28	6.87	7.39	0.99	2.03	2.31	2.53	2.79	2.89	.23	.26	.27	.24	.23	.28
13	3.39	4.97	5.73	6.30	7.57	7.41	1.39	2.01	2.24	2.53	2.85	2.99	.22	.24	.28	.24	.33	.24
14	3.37	5.07	5.67	6.20	6.86	7.00	1.39	2.04	2.25	2.48	2.75	2.82	.21	.25	.26	.25	.25	.24
15	3.65	5.08	5.65	6.23	6.91	7.07	1.48	2.04	2.25	2.51	2.75	2.83	.23	.24	.26	.24	.26	.25
16	3.26	5.25	5.89	6.35	6.93	7.36	1.36	2.06	2.28	2.52	2.83	2.92	.20	.27	.29	.26	.22	.26
16T†	3.02	4.49	5.16	5.67	6.26	6.58	1.22	1.87	2.09	2.25	2.52	2.63	.24	.20	.23	.26	.24	.25
17	2.77	4.56	5.51	6.21	6.52	7.12	1.13	1.87	2.14	2.45	2.64	2.81	.22	.22	.28	.26	.24	.26
18	2.89	4.88	5.47	6.24	7.01	7.33	1.16	1.92	2.19	2.49	2.79	2.84	.25	.27	.25	.26	.25	.29
18T†	2.26	4.87	5.42	6.00	6.49	6.88	0.93	1.92	2.18	2.42	2.68	2.77	.21	.27	.24	.24	.21	.24
19A	2.50	4.39	5.22	6.13	6.55	6.94	1.01	1.78	2.06	2.40	2.56	2.76	.23	.23	.27	.28	.28	.26
19B	2.89	4.88	5.77	6.19	6.88	7.04	1.19	1.90	2.22	2.54	2.71	2.82	.21	.28	.30	.22	.27	.25
19C	3.37	5.02	5.82	6.17	6.74	7.12	1.20	1.99	2.26	2.47	2.68	2.83	.40	.26	.29	.25	.26	.25
Type II Cements																		
21	2.33	4.56	5.65	6.22	6.96	7.33	0.92	1.83	2.17	2.48	2.77	2.86	.26	.24	.30	.26	.26	.28
21T†	2.32	4.64	5.49	6.05	6.80	7.21	0.96	1.88	2.17	2.43	2.75	2.86	.21	.23	.27	.24	.24	.26
22	2.75	5.24	6.01	6.49	7.31	7.50	1.14	2.00	2.38	2.57	2.83	2.97	.21	.31	.26	.26	.29	.27
23	2.99	4.94	5.72	6.43	6.93	7.15	1.20	1.92	2.24	2.54	2.74	2.85	.24	.28	.28	.26	.26	.25
24	3.25	4.94	5.43	6.10	6.48	6.77	1.28	1.93	2.16	2.43	2.57	2.66	.27	.28	.25	.26	.26	.27
25	2.93	4.50	5.44	6.11	6.79	6.97	1.21	1.81	2.14	2.38	2.66	2.78	.20	.24	.27	.28	.28	.25
Type III Cements																		
31	3.27	4.77	5.43	6.00	6.31	6.66	1.35	1.91	2.13	2.30	2.51	2.64	.21	.25	.27	.30	.26	.26
33	4.07	5.32	5.77	6.19	6.57	6.95	1.66	2.10	2.26	2.48	2.61	2.78	.23	.27	.28	.25	.26	.25
33T†	3.56	4.85	5.35	5.79	6.19	6.31	1.51	1.97	2.10	2.20	2.45	2.53	.18	.23	.27	.31	.27	.25
Type IV Cements																		
41	2.74	4.51	5.46	6.24	6.72	6.99	1.14	1.85	2.17	2.47	2.72	2.78	.20	.22	.25	.26	.23	.26
42	2.51	4.05	5.48	6.23	7.01	7.35	1.01	1.65	2.09	2.45	2.81	2.89	.24	.25	.32	.27	.24	.27
43	2.18	4.62	5.67	6.16	6.74	7.11	0.90	1.85	2.20	2.44	2.64	2.80	.22	.25	.29	.26	.28	.27
43A	2.56	4.27	5.41	6.20	6.81	7.42	1.02	1.71	2.15	2.49	2.70	2.93	.25	.25	.26	.25	.26	.27
Type V Cement																		
51	3.12	5.07	5.72	6.51	7.27	7.56	1.24	1.97	2.24	2.56	2.83	3.03	.26	.29	.28	.27	.29	.25

*Specimens: 6 x 6 x 30-in. beams. Cured continuously moist. Each result is the average of tests of three specimens.
†"Treated" cements. Comparable to present-day air-entraining cements.

TABLE 13—ELASTIC PROPERTIES (DYNAMIC) OF CONCRETES MADE WITH LTS CEMENTS, 4½ BAGS PER CU YD*

Cement	Young's modulus psi × 10^{-6}						Shear modulus, psi × 10^{-6}						Poisson's ratio					
	1 day	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years	1 day	7 days	28 days	3 months	1 year	3 years
Type I Cements																		
11	2.41	4.85	5.53	6.03	6.47	6.66	0.96	1.90	2.15	2.39	2.56	2.64	.25	.28	.29	.26	.27	.26
11T†	2.22	4.02	4.75	5.13	5.66	5.82	0.90	1.63	1.90	2.05	2.17	2.36	.24	.23	.25	.25	.30	.23
12	2.02	4.73	5.64	6.19	6.48	6.94	0.85	1.89	2.21	2.46	2.55	2.74	.19	.25	.28	.26	.27	.27
12T†	1.68	4.61	5.39	5.85	6.53	6.82	0.69	1.83	2.15	2.32	2.60	2.73	.21	.26	.26	.26	.26	.25
13	2.71	4.44	5.38	6.01	6.53	6.83	1.12	1.83	2.10	2.41	2.63	2.75	.21	.22	.28	.25	.24	.24
14	2.57	4.70	5.38	5.81	6.26	6.53	1.05	1.86	2.05	2.27	2.49	2.58	.22	.26	.31	.28	.26	.27
15	2.98	4.87	5.34	5.98	6.33	6.46	1.20	1.94	2.13	2.33	2.49	2.54	.24	.26	.25	.28	.27	.27
16	2.60	4.69	5.46	5.98	6.37	6.66	1.05	1.87	2.15	2.36	2.57	2.65	.23	.25	.27	.26	.24	.26
16T†	2.47	4.13	4.87	5.17	5.74	6.13	1.06	1.65	1.96	2.05	2.26	2.42	.16	.25	.24	.26	.27	.26
17	2.17	4.31	5.19	5.89	6.22	6.52	0.87	1.71	1.98	2.26	2.47	2.49	.25	.26	.31	.30	.26	.31
18	2.33	4.52	5.37	5.99	6.50	6.68	0.93	1.79	2.07	2.36	2.56	2.67	.25	.26	.30	.27	.27	.25
18T†	1.97	4.49	5.32	5.83	6.39	6.65	0.86	1.85	2.15	2.31	2.54	2.62	.15	.22	.24	.26	.26	.27
19A	1.84	3.86	4.88	5.64	6.13	6.61	0.74	1.55	1.91	2.18	2.43	2.57	.24	.24	.27	.29	.26	.29
19B	2.12	4.27	5.16	5.85	6.26	6.57	0.88	1.73	2.05	2.26	2.49	2.59	.20	.24	.25	.29	.26	.27
19C	2.68	4.64	5.49	5.92	6.43	6.66	1.14	1.87	2.17	2.31	2.51	2.62	.18	.24	.27	.28	.28	.27
Type II Cements																		
21	1.72	4.12	5.12	5.96	6.55	6.77	0.72	1.63	1.99	2.30	2.57	2.73	.19	.27	.29	.30	.28	.24
21T†	1.78	4.23	5.14	5.73	6.29	6.68	0.78	1.71	2.06	2.30	2.52	2.66	.14	.24	.25	.24	.25	.25
22	1.95	4.39	5.51	6.05	6.48	6.87	0.83	1.77	2.17	2.43	2.57	2.73	.17	.24	.27	.24	.26	.26
23	2.32	4.49	5.38	5.96	6.48	6.77	0.95	1.77	2.12	2.32	2.53	2.67	.22	.27	.27	.29	.28	.27
24	2.32	4.24	5.10	5.67	6.16	6.37	0.94	1.68	1.94	2.19	2.38	2.48	.23	.26	.31	.29	.29	.29
25	2.11	3.82	4.96	5.69	6.35	6.60	0.88	1.51	1.95	2.23	2.51	2.55	.20	.27	.27	.27	.26	.29
Type III Cements																		
31	2.88	4.78	5.22	5.79	6.18	6.40	1.19	1.89	2.07	2.25	2.45	2.49	.21	.26	.26	.28	.26	.28
33	3.69	4.93	5.43	5.96	6.26	6.57	1.50	1.94	2.16	2.33	2.50	2.57	.23	.27	.26	.28	.26	.28
33T†	3.33	4.47	4.97	5.43	5.83	5.93	1.35	1.84	1.97	2.13	2.31	2.39	.24	.22	.26	.28	.26	.24
Type IV Cements																		
41	1.92	3.74	4.95	5.86	6.35	6.69	0.80	1.50	1.93	2.24	2.50	2.62	.20	.25	.28	.30	.27	.28
42	1.90	3.52	4.89	5.81	6.61	7.14	0.83	1.42	1.92	2.29	2.59	2.79	.15	.24	.27	.27	.28	.28
43	1.63	3.87	5.20	5.86	6.30	6.79	0.64	1.60	2.08	2.27	2.50	2.69	.28	.21	.25	.29	.26	.26
43A	1.80	3.58	4.85	5.78	6.44	6.97	0.77	1.43	1.92	2.29	2.55	2.76	.17	.25	.25	.26	.26	.26
Type V Cement																		
51	2.41	4.36	5.37	6.10	6.61	7.12	0.97	1.74	2.15	2.46	2.66	2.85	.24	.25	.25	.24	.24	.25

*Specimens: 6 x 6 x 30-in. beams. Cured continuously moist. Each result is the average of tests of three specimens.
†"Treated" cements. Comparable to present-day air-entraining cements.

TABLE 14—ELASTIC PROPERTIES (DYNAMIC) OF CONCRETES MADE WITH LTS CEMENTS, 3 BAGS PER CU YD (AIR-ENTRAINED)*

Cement No.	Test results at age indicated					
	1 day	7 days	28 days	3 months	1 year	3 years
	Young's modulus, psi × 10^{-6}					
12	0.91	3.25	4.31	5.09	5.37	5.40
23	1.26	3.38	4.29	5.00	5.52	5.52
43A	0.92	2.38	3.80	5.18	5.73	5.95
	Shear modulus, psi × 10^{-6}					
12	0.41	1.33	1.72	1.94	2.10	2.12
23	0.54	1.35	1.72	1.92	2.13	2.17
43A	0.40	0.99	1.55	2.01	2.22	2.34
	Poisson's ratio					
12	0.12	0.22	0.25	0.31	0.28	0.27
23	0.16	0.26	0.25	0.30	0.30	0.28
43A	0.14	0.21	0.23	0.29	0.29	0.27

*Specimens: 6 x 6 x 30-in. beams. Cured continuously moist. Each result is the average of tests of three specimens.

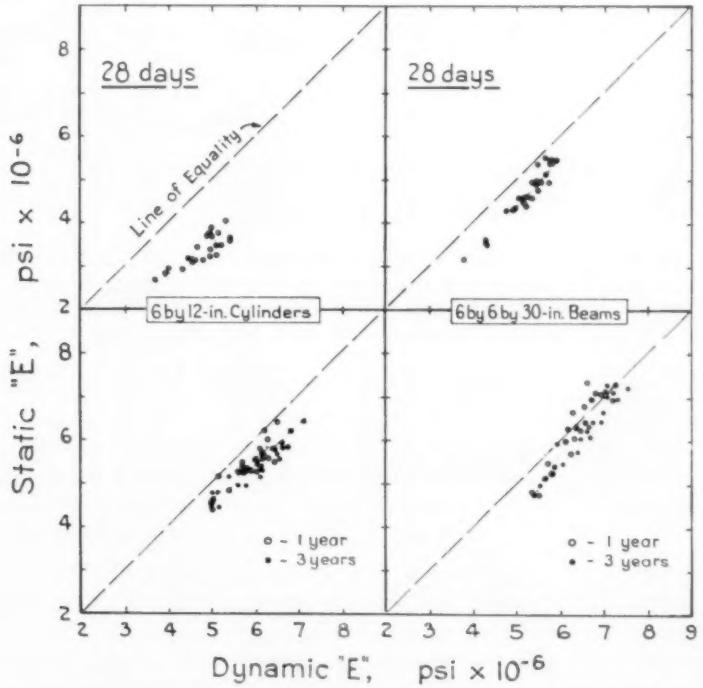

Fig. 7—Comparison of modulus of elasticity of concretes as determined by two different methods. Plotted points represent data shown in Table 15. Dynamic *E* by flexural vibration. Static *E* of 6 x 12-in. cylinders using compressometer, of 6 x 6 x 30-in. beams by deflection method

TABLE 15—COMPARISON OF DYNAMIC AND STATIC *E* DATA FOR CONCRETES MADE WITH LTS CEMENTS

Cement No.	Cement type	Young's modulus of elasticity, psi × 10^{-6}											
		6 x 12-in. cylinders						6 x 6 x 30-in. beams					
		Dynamic *E*			Static *E*			Dynamic *E*			Static *E*		
		28 days	1 year	3 years	28 days	1 year	3 years	28 days	1 year	3 years	28 days	1 year	3 years
6 bags per cu yd													
11	I	5.41	6.43	6.74	3.57	5.48	5.84	5.86	7.18	7.09	5.46	6.96	7.27
11T	IT*	4.64	5.38	5.61	3.14	4.86	5.00	5.04	5.85	6.06	4.60	5.41	5.48
15	I	5.30	6.28	6.74	4.02	5.57	5.86	5.65	6.91	7.07	5.52	6.99	7.10
19A	I	4.99	5.95	6.51	3.52	5.49	5.71	5.22	6.55	6.94	4.37	6.79	6.44
19B	I	5.20	5.97	6.60	3.47	5.56	5.64	5.77	6.88	7.04	5.39	7.07	7.11
19C	I	5.20	6.18	6.68	3.68	5.72	5.82	5.82	6.74	7.12	5.47	6.92	6.98
21	II	5.14	6.30	6.61	3.79	6.03	5.95	5.65	6.96	7.33	5.12	7.05	6.93
21T	IIT*	5.08	6.18	6.40	3.49	5.58	5.84	5.49	6.80	7.21	5.36	7.08	7.14
31	III	4.91	6.01	6.11	3.78	5.28	5.29	5.43	6.31	6.66	4.91	6.05	6.33
41	IV	4.98	6.16	6.65	3.88	5.67	5.83	5.46	6.72	6.99	4.97	6.96	6.68
51	V	5.43	6.50	7.11	3.65	6.41	6.50	5.72	7.27	7.56	4.96	7.29	7.22
4½ bags per cu yd													
11	I	5.09	5.90	6.18	3.26	5.29	5.47	5.53	6.47	6.66	4.96	6.23	6.30
11T	IT*	4.31	5.00	5.20	2.93	4.40	4.41	4.75	5.66	5.82	4.28	5.15	5.24
15	I	5.00	5.90	6.14	3.68	5.33	5.20	5.34	6.33	6.46	4.96	6.03	6.07
19A	I	4.62	5.70	6.16	3.13	5.47	5.59	4.88	6.13	6.61	4.31	6.00	6.26
19B	I	4.77	5.69	6.07	3.16	5.41	5.44	5.16	6.26	6.57	4.55	5.70	6.20
19C	I	4.93	5.92	6.14	3.39	5.53	5.24	5.49	6.43	6.66	4.75	6.24	6.31
21	II	4.65	6.08	6.45	3.45	5.79	5.80	5.12	6.55	6.77	4.61	6.42	6.46
21T	IIT*	4.56	5.82	6.09	3.09	5.31	5.37	5.14	6.29	6.68	4.47	6.66	6.10
31	III	4.84	5.61	5.83	3.69	5.28	5.00	5.22	6.18	6.40	4.60	6.26	5.75
41	IV	4.47	5.73	6.23	3.20	5.39	5.37	4.95	6.35	6.69	4.36	6.28	6.24
51	V	4.93	6.20	6.72	3.23	6.23	6.28	5.37	6.61	7.12	4.57	7.33	7.06
3 bags per cu yd (air-entrained)													
12	I	4.00	4.99	5.04	2.94	4.52	4.75	4.31	5.37	5.40	3.51	4.83	4.77
23	II	3.93	4.94	5.21	2.85	4.47	4.82	4.29	5.52	5.52	3.60	4.76	4.96
43A	IV	3.68	5.14	5.41	2.69	5.17	5.20	3.80	5.73	5.95	3.17	5.46	5.98

*"Treated" cements. Comparable to present-day air-entraining cements.

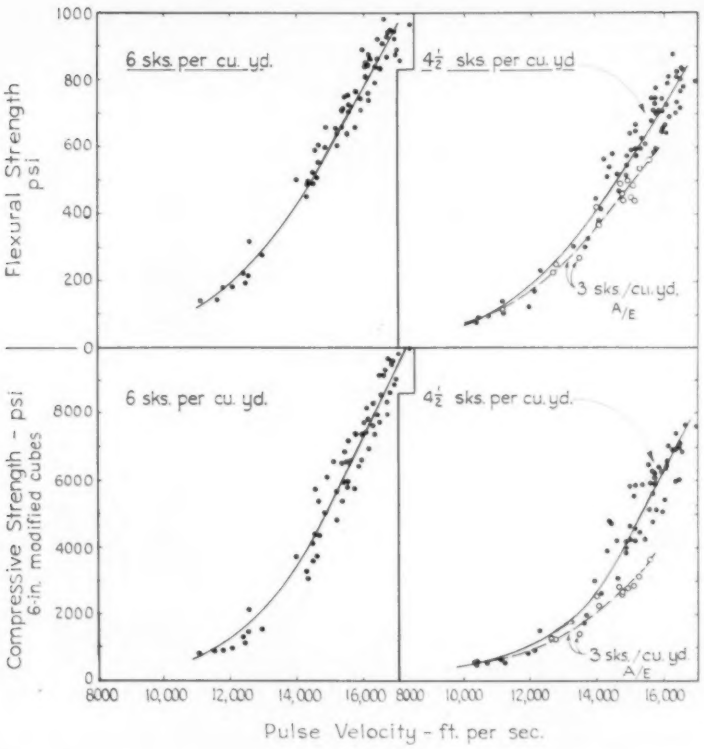
Flexural Strength psi
6 sks. per cu. yd.
4½ sks. per cu. yd.
3 sks./cu. yd. A/E
Compressive Strength - psi 6-in. modified cubes
6 sks. per cu. yd.
4½ sks. per cu. yd.
3 sks./cu. yd. A/E
Pulse Velocity - ft. per sec.

Fig. 8—Relationships between pulse velocity and strengths of concretes made with different types of cement. Plotted points represent data shown in Table 16. Specimens: 6 x 6 x 30-in. beams cured continuously moist. Pulse velocity: determined by soniscope, 30-in. path length. Test ages were 1, 7, and 28 days, 3 months, 1 and 3 years

Pulse velocities of concretes—Table 16 shows pulse velocity data from 1 day through 3 years determined on the concrete beams made for strength tests at 50 years. Specimens for 50-year tests were made with the cements specially selected for the elasticity tests (see Table 16).

Fig. 8 shows the relationships between pulse velocity and both flexural and compressive strengths of the concretes. The average relationships differ for the three cement contents, as was the case for the dynamic modulus-strength relationships. Here again, the changes are related to the changes in paste-aggregate ratio, as shown in Fig. 9. Apparently the pulse velocity through the aggregate is higher than that through the paste and consequently, at a particular strength level, the concrete containing a greater amount of aggregate per unit of paste exhibits a higher pulse velocity.

TABLE 16—VELOCITY DATA FOR CONCRETES MADE WITH LTS CEMENTS*

Cement No.	Cement type	Velocity (soniscope), ft per sec						
		1 day	3 days	7 days	28 days	3 months	1 year	3 years
6 bags per cu yd								
11	I	—	14,290	15,180	15,720	16,270	16,480	17,010
11T	IT†	11,770	13,350	13,970	14,850	15,340	15,720	15,890
15	I	—	14,200	14,880	15,530	16,160	16,410	16,630
19A	I	—	13,350	14,320	15,180	15,920	16,310	16,740
19B	I	12,440	13,990	14,650	15,500	16,060	16,480	16,930
19C	I	12,950	14,040	14,650	15,430	15,960	16,410	16,970
21	II	11,580	13,520	14,450	15,340	16,100	16,700	16,890
21T	IIT†	11,090	13,790	14,480	15,500	16,130	16,670	16,860
25	II	12,400	13,790	14,590	15,400	16,060	16,520	16,630
31	III	12,560	13,760	14,530	15,120	15,790	16,130	16,300
41	IV	12,040	13,590	14,310	15,530	16,130	16,590	16,820
51	V	12,540	13,890	14,530	15,530	16,270	16,780	17,360
4½ bags per cu yd								
11	I	—	13,690	14,590	15,340	15,850	16,130	16,380
11T	IT†	11,140	13,210	13,940	14,850	15,270	15,690	15,750
15	I	—	13,690	14,450	15,150	15,690	15,950	16,020
19A	I	—	12,400	13,640	14,650	15,430	15,950	16,340
19B	I	11,210	13,120	14,120	15,000	15,600	16,030	16,490
19C	I	12,100	13,660	14,310	15,120	15,720	16,100	16,490
21	II	10,380	13,070	14,050	15,000	15,720	16,370	16,600
21T	IIT†	10,420	13,210	14,040	15,150	15,750	16,340	16,490
25	II	11,130	12,800	13,710	14,880	15,690	16,100	16,230
31	III	12,290	13,610	14,340	14,970	15,560	15,920	16,060
41	IV	10,700	12,320	13,250	14,880	15,600	16,230	16,490
51	V	11,960	13,350	14,040	15,150	15,890	16,560	16,970
3 bags per cu yd (air-entrained)								
12	I	—	—	12,780	14,000	14,680	14,910	15,060
23	II	—	—	12,650	14,060	14,790	15,120	15,000
43A	IV	—	—	11,160	13,490	14,760	15,270	15,590

*Determinations on beam specimens made for 50-year strength tests. Path length, 30 in.
†"Treated" cements. Comparable to present-day air-entraining cements.

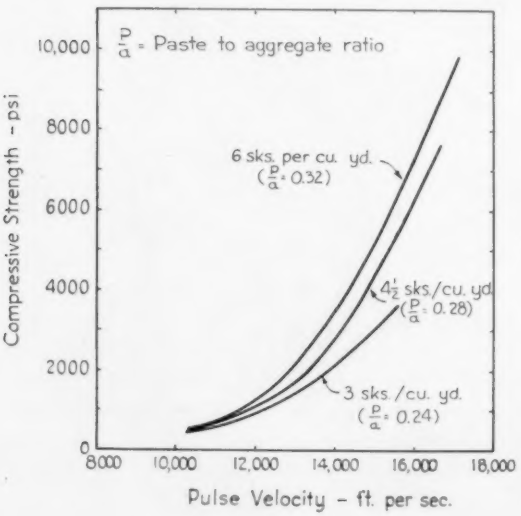

Fig. 9—Effect of paste-aggregate ratio on the relationship between pulse velocity and compressive strength. Average curves from Fig. 8

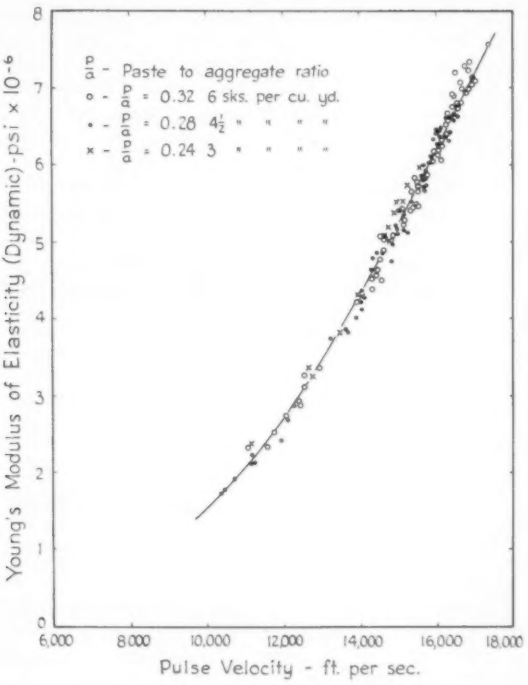

Fig. 10—Relationship between Young's modulus of elasticity (dynamic) and pulse velocity. Data from Tables 12, 13, 14, and 16

Fig. 10 shows the relationship between pulse velocity and dynamic (sonic) modulus of elasticity. For these concretes, continuously moist cured and made with aggregate from one source, there is an excellent relationship between these two elastic properties, regardless of changes in paste to aggregate ratio (changes in cement content), age of concrete, amount of air entrained, or type of cement. Changes in these variables affect both pulse velocity and dynamic modulus in the same manner.

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Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1958, for publication in the June 1958 JOURNAL.

Title No. 54-28

Investigation of Multibeam Bridges*

By RENÉ E. WALTHER†

SYNOPSIS

Following introductory reference to the load-carrying system of multibeam bridges (theory of orthotropic plates) the paper reports findings of test series on a large-scale model bridge of prestressed concrete. It is shown that such a bridge can accurately be analyzed as an orthotropic plate. The influence of factors which affect the behavior of the structure—slip between adjacent beams, interaction of shear keys, degree and location of lateral post-tensioning, etc.—have been studied. Empirical formulas for the relationship of the stiffness properties are derived.

INTRODUCTION

The broad development of precasting in the field of prestressed concrete has created a new type of structure, the so-called multibeam bridge, i.e., a bridge composed of simple beams placed side by side with the interaction between the members being provided by shear keys and lateral bolts, which may or may not be post-tensioned.

In the past 5 years, many such bridges have been successfully built in the United States with members of rectangular (solid or hollow) or channel-shaped cross sections. An exact physical interpretation of the manner in which such a bridge transmits an externally applied load to its supports is somewhat complex, because not only may the structure be nonhomogeneous and anisotropic, but also discontinuous (due to slip between adjacent beams).

These and other difficulties complicating the analysis, have often been eluded by assuming that a single bridge member would simply carry a more or less arbitrarily determined percentage of truck wheel loads. To meet safety requirements this percentage has to be extremely conservative, and consequently the economic benefits offered by such structures have not been fully utilized.

The problem of establishing a more realistic analysis has been the subject of an extensive investigation at Fritz Laboratory, Lehigh University; the work included a field test,[1] a study of the theory of orthotropic plates,[2] and a series of tests on a large-scale model bridge.[3] The latter is reported in this paper. Before dealing with these tests, however, some introductory reference to the theory of orthotropic plates shall be made.

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Notation

a = width of single beam
B = ½ width of bridge
EI_x = longitudinal bending stiffness
EI_y = lateral bending stiffness
F = total post-tensioning force
h = depth of single beam
I_y = lateral moment of inertia
L = length of bridge
m = number of individual beams
M_x = longitudinal bending moment
M_y = lateral bending moment
M_{yx} = twisting moment
M_{xy} = twisting moment
n = number of wheel loads over one cross section
P = applied concentrated load (P_e = edge, P_c = center)
Q_x = longitudinal shear force
Q_y = lateral shear force
s = denotes coefficient
w = deflection
α = EI_y/EI_x
β = coefficient of torsional rigidity

THE MULTIBEAM BRIDGE AS AN ORTHOTROPIC PLATE

The possible heavy load concentrations on modern bridges make it desirable to seek an optimum degree of interaction between the individual bridge members, with the effect of having all the beams participating in carrying such concentrated loads. Shear keys and lateral bolts (loose or post-tensioned) can provide efficient interaction, therewith generating a special kind of plate, i.e., an orthotropic plate. It is called orthotropic because the bending stiffness in the longitudinal direction $(EI)_x$ is different from the one in the lateral direction $(EI)_y$.*

One can distinguish two limiting cases: if the degree of lateral post-tensioning is very high, EI_y may rise to the same magnitude as EI_x (isotropic plate); if the lateral bolts are loose or hand tight, EI_y approaches zero (articulated plate).

The differential equation of an orthotropic plate can be written in the following form:

$$\frac{\partial^4 w}{\partial x^4} + 2\beta \frac{\partial^4 w}{\partial x^2 y^2} + \alpha \frac{\partial^4 w}{\partial y^4} = \frac{P(x,y)}{EI_x}$$

where $\alpha = EI_y/EI_x$ and 2β = coefficient of torsional rigidity.

Without giving a detailed solution of this differential equation, we only mention that it is possible to determine the deflections w and the internal forces under a given loading $P(x, y)$ by the use of three parameters.[2]

$$\frac{B}{L} = \frac{\text{half the bridge width}}{\text{total bridge length}}$$

$$\frac{a}{h} = \frac{\text{width of one beam}}{\text{depth of one beam}}$$

$$\alpha = \frac{EI_y}{EI_x} = \frac{\text{lateral bending stiffness}}{\text{longitudinal bending stiffness}}$$

*Since the modulus of elasticity can be considered independent of the direction, $(EI)_x$ and $(EI)_y$ can be replaced by EI_x and EI_y.

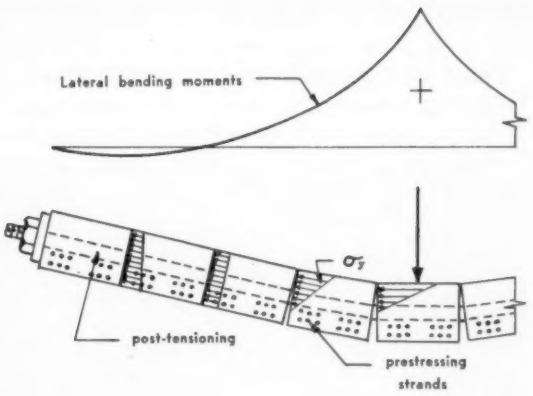


Fig. 1—Distribution of concrete stresses along a bridge cross section

The first parameter is merely dependent on the geometry of the bridge; the second is a function of the cross section of the individual beams. It replaces the coefficient of torsional rigidity which occurs in the differential equation. The relationship of a/h and the torsional rigidity was found by theoretical considerations which had to be verified by our tests.

The derivation of the parameter α is somewhat difficult, and no theoretical approach has yielded satisfactory results since the lateral bending stiffness is by no means a constant. It varies not only from point to point in the bridge, but it is also dependent on the magnitude and location of the concentrated load. This can be visualized by considering a transverse strip taken out of the bridge. It consists of rectangular blocks with no mutual cohesion; the lateral bending stiffness is provided only by the post-tensioning (Fig. 1). In practice, this post-tensioning force is hardly large enough to prevent small openings at the bottom of the joints. In these joints, the stress distribution becomes similar to one of a cracked cross section of a prestressed beam, i.e., this cross section is nonhomogeneous. The moment of inertia of a nonhomogeneous section is variable under the combined action of moments and normal forces (post-tensioning), making it impossible to derive α theoretically. This is one of the main reasons why an experimental investigation was necessary. It yielded simple empirical formulas for an average α value over the entire bridge, as a function of the load and the total post-tensioning only.

If slip occurs between adjacent beams, the problem of what should be called the lateral bending stiffness becomes more complex. Due to this discontinuity (slip), deflections and stress distribution do not follow the rules of the plate theory at all. The idea of compensating for the effect of slip by reducing the lateral bending stiffness would lead to a contradiction, because in the case of an articulated plate, where a development of slip is very likely, the lateral bending stiffness is zero and cannot be reduced. It was therefore necessary to evaluate the change of the internal forces by empirical approximations (explained later in this paper).

DISTRIBUTION OF INTERNAL FORCES AND DEFLECTIONS OF ORTHOTROPIC PLATES

The design of multibeam bridges is governed principally by the longitudinal bending moments M_x. Hence, the most important characteristic of such a structure is the lateral distribution of these moments over the cross section of the bridge, often referred to as "lateral load distribution." In a plate however, the load is not only transmitted by the longitudinal bending moments M_x, but also by the lateral bending moments M_y, the twisting moments M_{xy} and M_{yx}, and the shear forces Q_x and Q_y. As Fig. 2 illustrates, their distribution over a cross section is quite different. (This example is for the particular case of center loading where $\alpha = 0.5$, $B/L = 0.5$, $a/h = 1.0$.)

Since internal forces cannot be measured directly, it is necessary for test purposes to relate them theoretically to the deformations, which can be obtained directly from tests. This was done by using the theory of orthotropic plates.

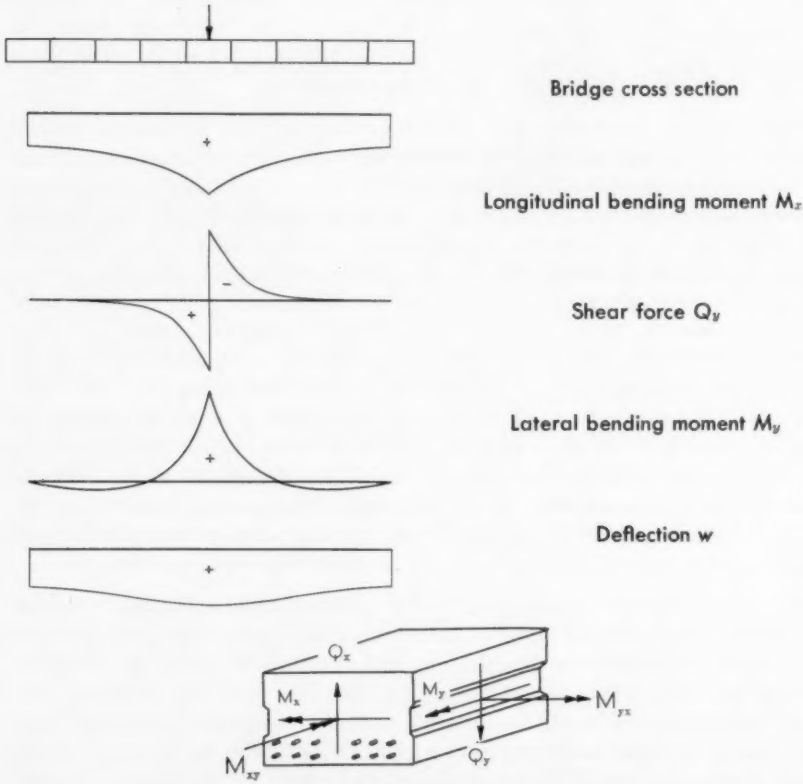


Fig. 2—Distribution of moments, shear forces, and deflections at the midspan cross section (center loading)

Of special interest is the relationship between longitudinal bending moments and deflections. It should be noticed that these moments are not proportional to the deflections. Therefore, the distribution curves for the moment coefficients* and the deflection coefficients are not identical. This can be visualized by considering the following: with a concentrated load at the center point of

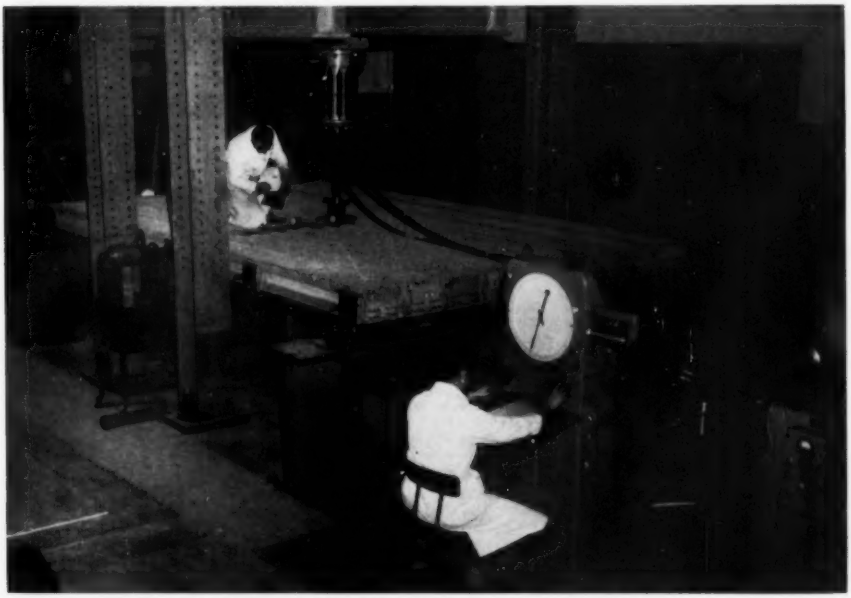


Fig. 3—Test set-up of the laboratory bridge

the bridge, the deflected shape of an edge beam is somewhat similar to the shape of a uniformly loaded beam, whereas, the middle beam acts more or less like a simple beam under a single concentrated load. The moment-deflection ratio for these two cases differs by 25 percent. Another explanation may be found in the theory of thin plates, where under a point load, the maximum deflection is finite; yet the maximum moment theoretically approaches infinity. In the practical case of a multibeam bridge, these effects lead to a deviation between the coefficients of the longitudinal bending moments and the deflection coefficients which may be as high as 50 percent.

THE TEST PROGRAM

General

In order to clarify the problem pointed out in the foregoing, a model bridge was built which was large enough to give a reliable comparison with an actual structure, but limited to some extent by the available facilities. The bridge of nine beams, with which the majority of tests were performed, had a 16-ft span and a width of 10 ft 9 in. (Fig. 3). Prestressing was applied

*It is convenient to present distribution coefficients, rather than the actual moments, shear forces, or deflections themselves. The coefficient for a particular point is defined as the ratio of the moment (shear force or deflection) at this point to the average moment (shear force or deflection) of the entire cross section. These coefficients are dimensionless and—in the range of elastic deformation—independent of the amount of load.

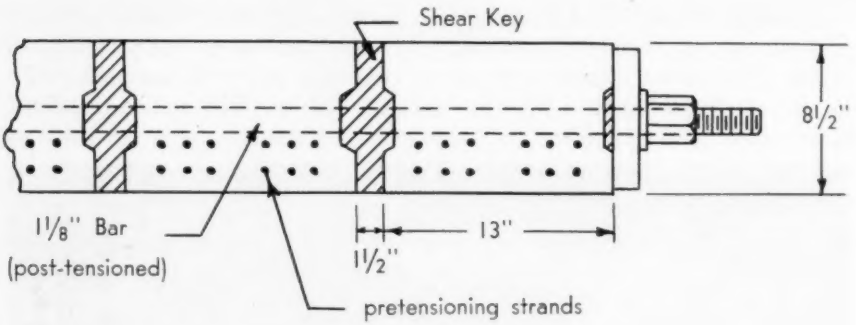


Fig. 4—Beam cross section

in two directions: longitudinally by means of pretensioning the twelve 5/16-in. strands in each beam; laterally by post-tensioning with five 1⅛-in. bars, located at the center, the quarter points, and over the supports of the bridge. This system was convenient because the lateral post-tensioning could easily be varied during the course of the tests to any desired degree. The cross section of individual beams forming the bridge is shown in Fig. 4.

To study the transfer of the shear forces, large keyways were put along the sides of the beams; in the first test series the keyways were filled with wood so that only friction participated in this transfer, whereas real shear transfer was realized in the second series by means of grouted concrete shear keys. Because of the initial warping of the beams, additional concrete had to be placed between adjacent beams (spaced 1½ in.) to guarantee a full bearing area for the lateral post-tensioning.

All tests were performed in the newly completed test bed of the Fritz Engineering Laboratory (Fig. 3). The loads were applied by the Amsler equipment (hydraulic jacks fed by a pendulum manometer).*

The deflections were measured at the center point and at the quarter points of each bridge member by means of Ames dial gages (sensitivity 0.001 in.) mounted on measuring frames. For the final destruction test, the dial gages were replaced by scales, in order to permit measurements up to the ultimate load.

Answering the important question, "Does slip occur and if so, how much?" required a precise measuring device, because even a small slip has a decisive effect on the distribution of deflections and internal forces. This high precision was achieved by measuring directly the relative slip between two adjacent beams (Fig. 5); two dial gages (sensitivity 0.0001 in.) were

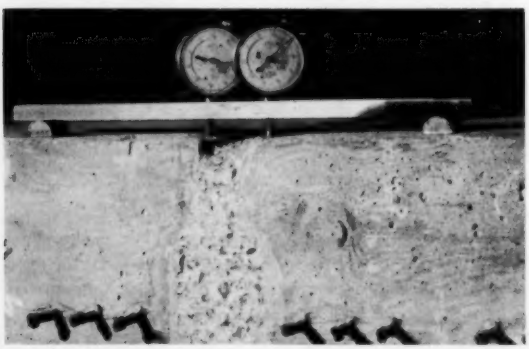


Fig. 5—Equipment to measure slip between adjacent beams. Note grouted shear key

*A detailed description of this test equipment can be found in a paper recently completed at Fritz Laboratory.[4]

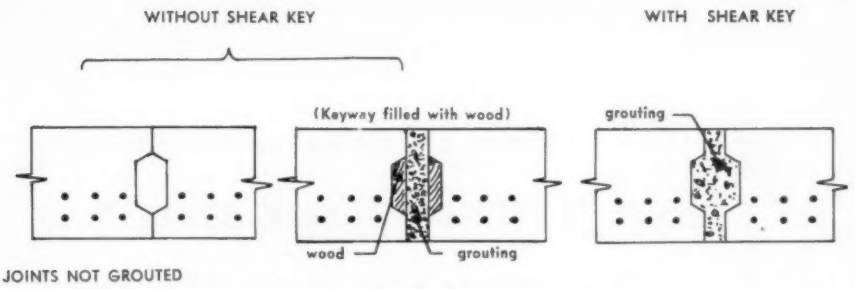


Fig. 6—Variation of shear transfer

fixed on a rectangular bar which spanned the two beams from center to center. Small half round bars, fixed with sealing wax to the beams, served as a support for the device. The points of the dials touched two small bearing plates on both sides of the shear keys.

Details of test program

The 58 different tests, performed over a period of 3 months in a sequence dictated by the operations they involved, can be divided into three major categories, which are:

1. Tests to determine the stiffness properties of the bridge:
   a. The properties of the individual beams
   b. The longitudinal bending stiffness of the bridge
   c. The lateral bending stiffness of the bridge
   d. The torsional rigidity of the bridge
2. Tests to investigate the influence of:
   a. Degree of lateral post-tensioning
   b. Location of lateral post-tensioning
   c. Location of the external load
   d. Interaction of shear keys
   e. Slip between adjacent beams
3. Tests to determine the lateral load distribution in the inelastic range up to destruction of the bridge.

Table 1 gives description, numbers, and some important results of the tests.

First, an individual beam was tested to destruction, in order to find the elastic and plastic deflections, the cracking load, and the ultimate load of the bridge members.

Tests 70 to 82 to determine the elastic properties of the entire bridge were all performed with grouted shear keys, varying only the post-tensioning force of the bars.

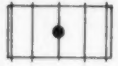
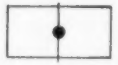
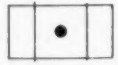
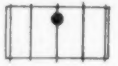
In Tests 80, 81, and 82, yielding the longitudinal bending stiffness EI_z, the bridge was supported across the ends, the jack load being distributed to a line load by a heavy I-beam. The lateral bending stiffness was found similarly in Tests 70 to 72 with longitudinal supports along the two edge beams and a line load along the center beam.

During Tests 75 to 77 for determination of the torsional rigidity the bridge was in an indifferent state of equilibrium; two diagonally related corners were loaded, the other two supported. The load was transmitted to the corners by an I-beam, supported by half-round blocks at the other two corners; the reactions were taken by spherical bearings.

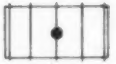
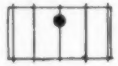
The investigation of factors which affect the lateral load distribution on the bridge was the essence of this study (Tests 3 to 65).

Variation of shear transfer was studied in two major groups: tests without acting shear keys, and tests with shear keys (Fig. 6). With the exception of Tests 3 to 6 (determination of the influence of the ratio B/L) the two groups involved the same variables, namely the degree and location of lateral post-tensioning and the location of the concentrated load. Only the two load conditions which can yield maximum moments were considered, i.e., a load at the

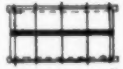
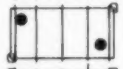
TABLE 1—TESTS PERFORMED AND RESULTS*

Description of tests	Test No.	Figure	Lateral post tensioning (tons/bar)					Load (Kips)	α	Slip (max.) $s_{w\ max.}$	Slip (max.) $s_{M\ max.}$	Slip (max.) 1/10000"
I. TESTS WITHOUT SHEAR KEY												
A. Variation of ratio $\frac{B}{L}$ $\frac{\frac{1}{2}\text{ width of bridge}}{\text{Length of bridge}}$												
	3		30	30	30	30	30	10	0.731	1.005	1.106	
Tests with 5 beams	4		10	10	10	10	10	10	0.439	1.023	1.168	
	5		10	30	30	30	10	10	0.727	1.013	1.188	
Tests with 7 beams	6		10	10	10	10	10	10	0.389	1.055	1.304	
B. Variation of degree of Post-tensioning	10		10	30	30	30	10	30	0.65	1.23	1.483	74
	11		10	20	20	20	10	30	0.58	1.28	1.570	76
	12		10	10	10	10	10	30	0.50	1.33	1.645	114
	13		5	5	5	5	5	30	0.38	1.37	1.725	166
	14		2.5	2.5	2.5	2.5	2.5	30	0.27	1.42	1.819	277
	15		1	1	1	1	1	30	0.20	1.49	1.922	199
C. Influence of the position of the post-tensioning bar	20		0	0	30	0	0	30	0.34	1.43	1.819	83
	21		0	0	20	0	0	30	0.22	1.56	2.015	103
	22		0	0	10	0	0	30	0.10	1.64	2.148	87
	23		0	0	5	0	0	30	0.095	1.77	2.349	254
	25		0	30	0	30	0	30	0.50	1.31	1.625	91
	26		0	20	0	20	0	30	0.34	1.42	1.800	97
	27		0	15	0	15	0	30	0.34	1.43	1.818	57
	28		0	10	0	10	0	30	0.34	1.42	1.800	86
	29		0	5	0	5	0	30	0.20	1.49	1.925	93
	32		15	0	0	0	15	5	0.09	1.52	1.993	32
	33		10	0	0	0	10	5	0.09	1.60	2.099	38
	34		5	0	0	0	5	5	0.09	1.60	2.099	....
D. Influence of position of concentrated load	40		10	30	30	30	10	10	0.28	1.87	2.730	5
	41		10	10	10	10	10	10	0.10	1.96	2.840	12
	42		5	5	5	5	5	10	0.10	1.99	2.890	15
Tests with edge load	43		1	1	1	1	1	10	0.19	2.15	3.120	29
E. Tests without shear keys or grouting	45		5	5	5	5	5	5		1.82		1825
	46		5	0	5	0	5	5				885
	47		0	0	5	0	0	5		5.16		1453
	48		5	0	0	0	5	5		2.86		1438

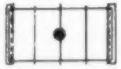
II. TESTS WITH SHEAR KEYS

A. Variation of degree of post-tensioning (center loading)	50		30	30	30	30	30	30	0.65	1.030	1.244	1
	51		10	10	10	10	10	30	0.50	1.060	1.302	5
	52		5	5	5	5	5	30	0.38	1.070	1.349	6
	53		1	1	1	1	1	30	0.20	1.093	1.415	9
	54		0	0	0	0	0	30	0.08	1.135	1.506	14
	55		0	0	30	0	0	30	0.34	1.073	1.362	5
	56		0	0	10	0	0	30	0.10	1.105	1.454	10
	57		0	0	5	0	0	30	0.095	1.121	1.488	10
	58		0	0	1	0	0	30	0.042	1.135	1.506	12
	59		10	0	0	0	10	30	0.09	1.120	1.465	14
B. Influence of position of loading (edge loading)	60		30	30	30	30	30	20	0.28	1.63	2.358	0
	61		5	5	5	5	5	20	0.26	1.64	2.371	0
	62		0	0	0	0	0	20	0.10	1.71	2.484	0
	63		0	0	5	0	0	20	0.10	1.69	2.484	0
	64		0	0	30	0	0	20	0.19	1.66	2.424	0
	65		30	0	0	0	30	20	0.19	1.67	2.424	0

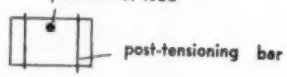
III. TESTS TO DETERMINE THE ELASTIC PROPERTIES OF THE ENTIRE BRIDGE

Lateral bending stiffness	70		10	30	30	30	10	
	71		5	5	5	5	5	
	72		1	1	1	1	1	
Torsional stiffness	75		10	30	30	30	10	
	76		5	5	5	5	5	
	77		1	1	1	1	1	
Longitudinal bending stiffness	80		10	30	30	30	10	
	81		5	5	5	5	5	
	82		1	1	1	1	1	

IV. TEST TO FAILURE

90		5	5	5	5	5	30	0.10	1.109	1.448
90		5	5	5	5	5	60	0.07	1.156	1.508
90		5	5	5	5	5	90	0.068	1.160	1.512
90		5	5	5	5	5	110	0.02	1.227	1.590
90		5	5	5	5	5	113.4	FAILURE		

*Notation: $s_{M\,max}$ = maximum value of the coefficient of lateral moment distribution
$s_{w\,max}$ = maximum value of the coefficient of lateral load deflection
α = EI_y/EI_z



center point of the bridge and a load at midspan of the edge beam. The post-tensioning was varied by choosing different forces in the bars as well as by changing the number of bars.

Finally the lateral load distribution was studied in the inelastic range of deflection. This test (No. 90), up to destruction, was performed with 5 kips post-tensioning on each of the five bars and with a concentrated load at midspan.

TEST RESULTS

Rather than present all test results involving about 30,000 readings, an effort will be made to show relationships between the different influencing factors.

Stiffness properties of the bridge

Individual beams—The bending stiffness of an individual beam before cracking, was 25×10^8 lb-in.^{2} In order to compare this value with the bending stiffness of the bridge—in plate problems always given per unit width—it had to be divided by the width of the beam (13 in.), i.e.,

$$EI_x = 196 \times 10^6 \text{ lb-in.}^2/\text{in.}$$

Longitudinal and lateral bending stiffness of the bridge—Due to the effect of the lateral contraction (Poisson's ratio) and the fact that the joints of the beam were grouted, the longitudinal bending stiffness per unit width of an individual beam differed from longitudinal bending stiffness of the entire bridge; furthermore, the latter depends on whether or not one includes the grouting in the total bridge width. Longitudinal bending stiffness of the entire bridge is:

grouting excluded $EI_x = 205 \times 10^6$ lb-in.^{2}/in.
grouting included $EI_x = 185 \times 10^6$ lb-in.^{2}/in.

In both cases, single beam and bridge, the longitudinal bending stiffness could be considered as constant up to the cracking load.

This did not hold true for the lateral bending stiffness since the post-tensioning could not prevent openings between adjacent beams; thus the bending stiffness was dependent on the magnitude of the load. Fig. 7 shows the large variation of EI_y as the load increases.

In order to derive the factor $\alpha = EI_y/EI_x$ from these tests, the lateral bending stiffness was taken at the point where the curve of Fig. 7 becomes horizontal, yielding the following ratios.

Post-tensioning per bar, tons	α for line loading	α for concentrated center loading
30	0.41	0.65
5	0.017	0.38
1	0	0.20

These α values, however, are much too small for the same bridge subjected to concentrated loads, because in this case the maximum lateral bending moments are confined to a small area under the load whereas they extend over the whole length of the bridge under line load. The experimental α values for concentrated loads (Tests 50, 52 and 53) are also given in the above table.

For this reason, the way of finding α by separately bending the bridge longitudinally and laterally was abandoned in favor of deriving it directly from the deflections of the normal loaded and supported bridge.

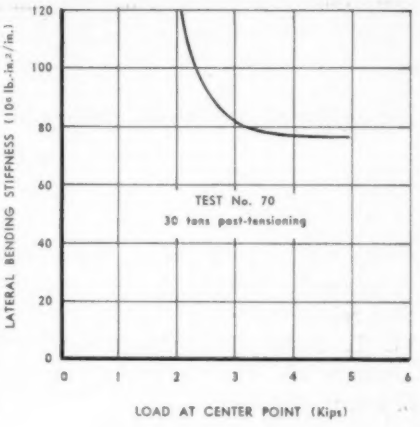


Fig. 7—EI_y versus load

Torsional rigidity of the bridge—Under the loading conditions for study of torsional rigidity (Fig. 8) no bending moments M_x and M_y occur; the only existing internal moments acting on an element $dx\,dy$ are the twisting moments M_{xy} and M_{yx}.

The deflections follow the formula:

$$w = -\frac{1}{EI_x}\frac{M_{xy} + M_{yx}}{2\beta}xy$$

This expression is linear in x and y; thus the deformed bridge has two straight lines as generatrices, the x and y axis. This was confirmed in Tests 75 and 76, for all beams and strips perpendicular to the beams remained approximately straight. Fig. 8 shows a perspective view of the shape of the deformed bridge, and the deflections of Test 75 at the load of 10,000 lb at both load points.

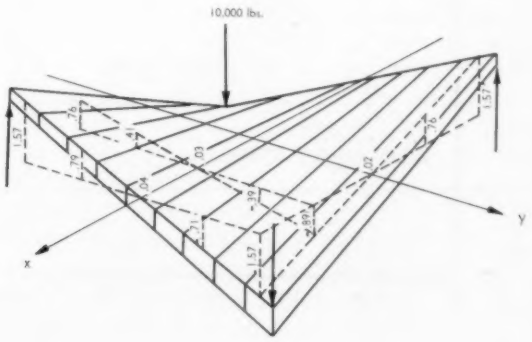


Fig. 8—Torsional rigidity, Test 75. Deflections shown are in in.

Since in our case the twisting moments can be expressed by the load

$$M_{xy} + M_{yx} = P$$

the coefficient of torsional rigidity becomes

$$2\beta = -\frac{P}{wEI_x}xy$$

which can be read directly from our deflection data.

As may be recalled the relationship between α and β for gridworks and two-way reinforced concrete slabs gave rise to various assumptions, notably by M. T. Huber[5,6] and Massonnet.[7] Since these assumptions proved to be incompatible with theoretical considerations for orthotropic plates, A. Roesli[2] proposed the formula

$$\beta = 3K(1 - \alpha^{3/2}) + \alpha$$

where K is a numerical factor for torsion of rectangular beams, depending on a/h.[8]

In Fig. 9 the results of Tests 75 and 76 are plotted together with some of the mentioned assumptions, all reduced for the particular case of the test bridge.

The correlation between the experimental and theoretical results indicates that under the various assumptions proposed for this relationship, the one established by A. Roesli comes closest to reality. The latter will be used throughout this paper.

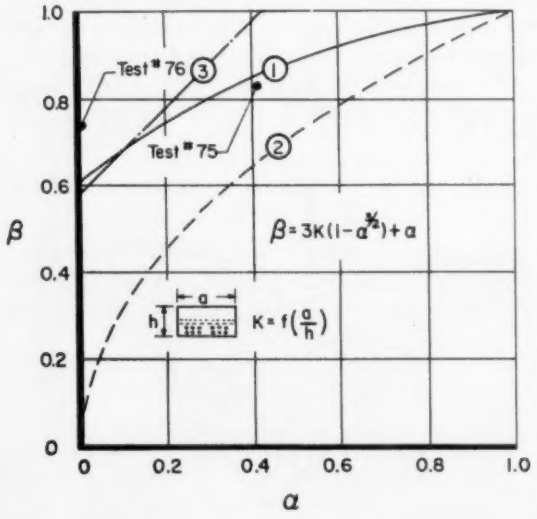


Fig. 9—Relationship between α and β. a/h 1.5. Solid dot shows measured relationship of Test 75. Curve 1 is theoretical relationship assumed by Roesli for orthotropic plates. Curve 2 is Huber's assumed relationship for two way slabs ($\beta = \sqrt{\alpha}$). Curve 3 is theoretical relationship generally applied for grids

Comparison between theory and tests (deflection and moment distribution)

The most conclusive data in a plate investigation are the deflections; if the deflected shape of the bridge is known in all details, all internal forces and reactions can be found by differentiation. Therefore, one could also determine the important longitudinal bending moment M_x by differentiating twice the measured deflections, according to the formula

$$M_x = -EI_x \frac{\partial^2 w}{\partial x^2}$$

(effect of Poisson's ratio neglected). This method would require a very large number of measuring points and would not be sensitive enough to indicate possible moment concentrations.

More adequate is a consideration of the lateral deflection distribution which, based on the theory of orthotropic plates, can yield the longitudinal bending moments far better than the method above. First, however, it had to be proved that this theory applies with sufficient accuracy.

For this proof only those tests which fulfilled the basic assumption that no slip occurs between adjacent beams could be considered. This was the case in all tests with grouted shear keys. A certain difficulty developed from the fact that one of the three parameters of the problem, i.e., α, cannot be found by theoretical considerations or by special tests, for reasons explained before. Therefore, distribution of the lateral deflection coefficients was calculated as a function of α; the other two parameters, B/L and a/h, are constants for a given bridge. By varying α, the theoretical curve with the same maximum deflection coefficient as the experimental one could be found and the correlation between the distribution of measured and theoretical coefficients could be checked. Fortunately, the correlation was excellent throughout the test series and it can be concluded that the theory is well applicable as long as little or no slip occurs. This is documented in Fig. 10.

For the two cases without any post-tensioning α was small but not zero as expected. When the bridge was disassembled the reason for this was found; the grout had adhered so strongly to the beam that it had provided considerable lateral bending stiffness. Based on these findings, the longitudinal bending moments—or more precisely their distribution coefficients—were calculated according to the theory of orthotropic plates. They are also given in Fig. 10.

If slip between adjacent beams develops or if the interaction of the shear keys is incomplete, no direct correlation between the theory and the test results exists. A method had to be established to safely approximate the longitudinal bending moments for each beam, necessary for the design. This method is based on the following assumption: the slip does not change the lateral bending stiffness of the bridge. This means that the deflection of a lateral strip is considered to consist of two separate phenomena—one is a continuous rotation of the strip elements (i.e., simple bending); the other is a

discontinuous vertical displacement of some strip segments (a strip segment = a cross section of an individual beam). Action of the plate, therefore, can also be imagined to consist of two separate phases: in the first, the bridge deflects as a normal orthotropic plate (plate bending); in the second, caused by slip, the longitudinal beams deflect individually without torsion (additional beam bending). The additional beam bending can be positive or negative depending upon proximity to the load. The longitudinal bending moments

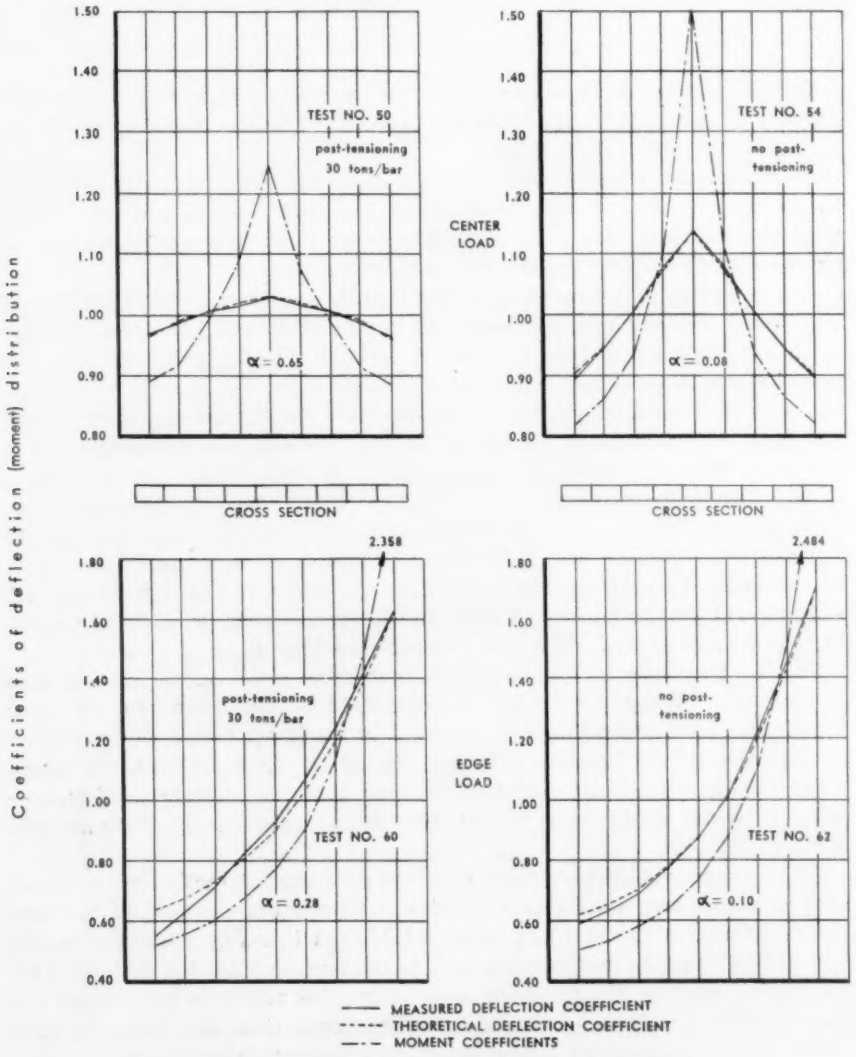


Fig. 10—Theoretical and measured deflection distribution

resulting from plate bending follow the same rules used so far. The percentage change in these bending moments is considered proportional to the percentage change in the deflection due to slip. Expressed in terms of coefficients, this becomes

$$S_{M_s} = \frac{S_M}{S_w} \times S_{W_s}$$

S_{M_s} = moment coefficient with slip
S_M = moment coefficient without slip
S_{W_s} = deflection coefficient with slip (measured)
S_w = deflection coefficient without slip (measured)

It can be proved that this method overestimates the maximum bending moment and is, therefore satisfactory for design purposes.

Influence of *B*/*L* ratio

The less the relative width of a bridge, the better is the lateral load distribution, i.e., the smaller are maximum moment and deflection coefficients.

The relationship between these coefficients and the B/L ratio could be determined from Tests 3 to 15, where the width of the bridge was varied by consecutively assembling five, seven, and nine beams. In Fig. 11 two typical curves are plotted for post-tensioning forces of 10 and 30 tons per bar, with a

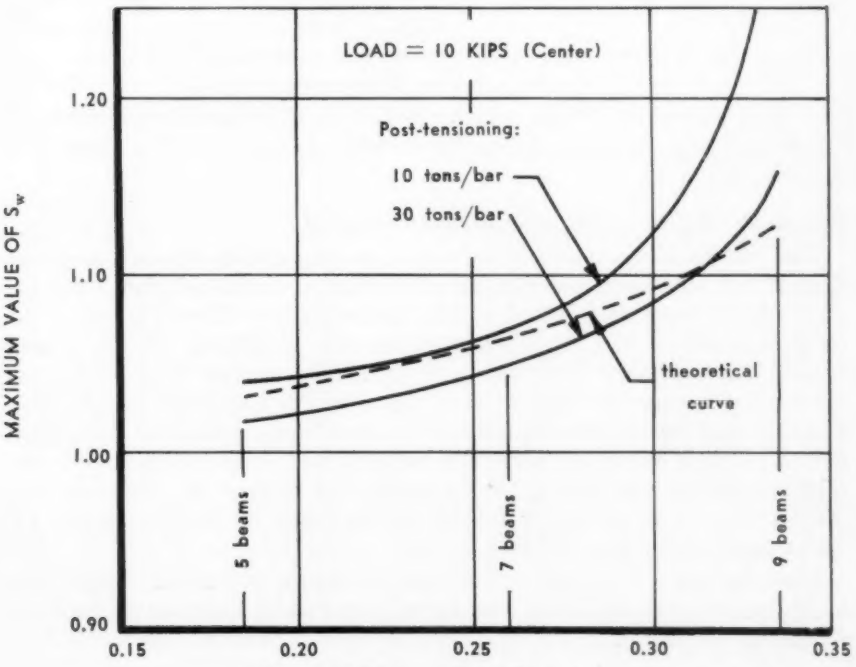


Fig. 11—Influence of the *B*/*L* ratio

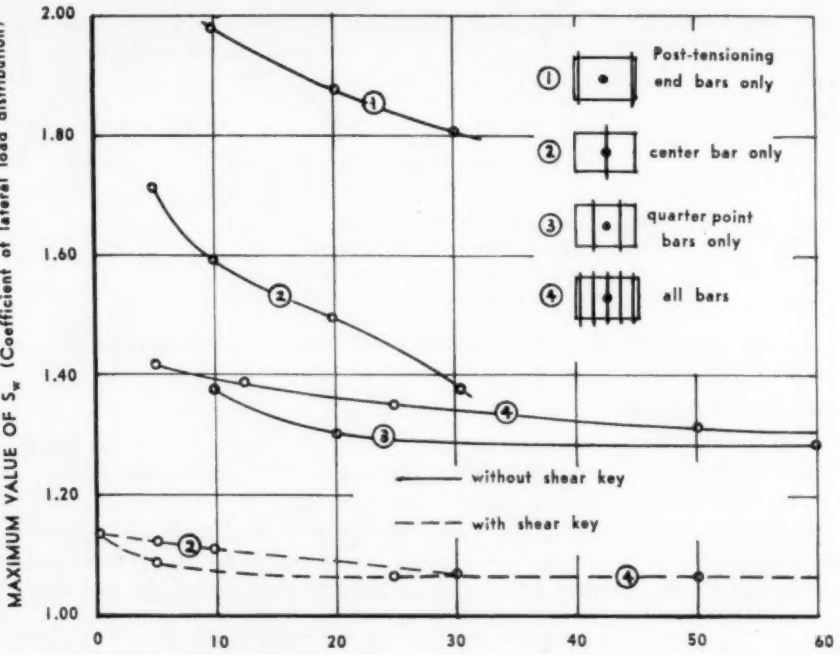


Fig. 12—Total post-tensioning force versus maximum value of S_w

load of 10 kips at the bridge center. Deviation of the measured curve from the theoretical is the result of slip, which could not be prevented in these tests without shear keys.

Influence of degree and location of post-tensioning

The maximum moment and deflection are dependent not only upon the degree* but also upon the location of the lateral post-tensioning. The influence of the latter was studied by post-tensioning four different bar combinations as shown in Fig. 12, where the total post-tensioning force is plotted against the maximum deflection coefficient. The smaller these coefficients are for a given post-tensioning force, the more efficient is the combination. The best load distribution was achieved by post-tensioning only at the quarter points. This is somewhat surprising because, analogous to a gridwork, one might expect post-tensioning of the center bar to give the optimum load distribution. A plausible explanation can be found by considering the different shear characteristics for each case.

A smaller but still considerable contribution to the lateral load distribution is obtained by post-tensioning over the supports, which increases the torsional rigidity of the bridge.

*We define the degree of lateral post-tensioning as the total lateral post-tensioning force of the bars divided by he lateral surface of the bridge.

From Fig. 12 it can also be concluded that a good shear transfer is far more effective than a high lateral post-tensioning. Strong shear keys are therefore of prime importance.

Influence of load magnitude (destruction test)

Since the lateral bending stiffness of a bridge depends on the magnitude of the applied load, the moment and deflection coefficients cannot remain constant. Only for $\alpha = 1.0$ (isotropic plate), and even then only so long as the resulting strains stay in the elastic range, are these coefficients independent of the load.

The magnitude and significance of the variation in these deflection coefficients was observed in the final destruction test (post-tensioning 5 tons at each bar). Variation of the maximum deflection coefficient as a function of load is plotted in Fig. 13; the irregular shape of this curve was a result of the consecutive cracking of individual longitudinal beams. Notice that $S_{w\ max}$ increased only 2.5 percent in the range from zero load up to cracking of the first beam. The total change in $S_{w\ max}$ amounted to about 14 percent.

The bridge finally failed at a load of 113.4 kips by punching shear. Of special interest is the fact that the ultimate capacity of the bridge was only 4 percent smaller than the sum of the ultimate capacities of all the beams. This means that for an actual bridge, where such load concentrations are impossible, the ultimate capacity of the entire structure can be obtained by adding the capacities of all the members.

EMPIRICAL FORMULAS

The parameter α

Since theoretical prediction of α seems impossible, an attempt was made to find a method for its approximation. Hence, empirical formulas were developed in terms of post-tensioning and load. For these formulas, all details such as the number or location of the post-tensioning bars were neg-

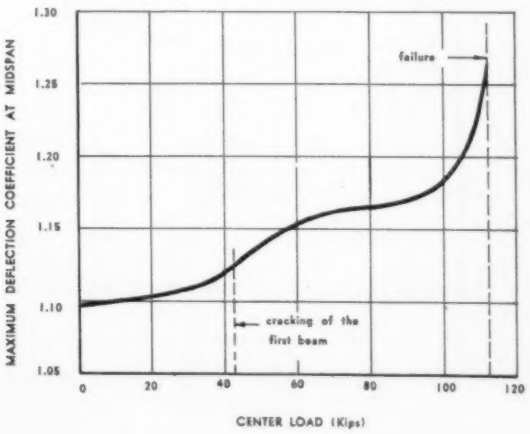


Fig. 13—Variation of maximum deflection coefficient as a function of the load

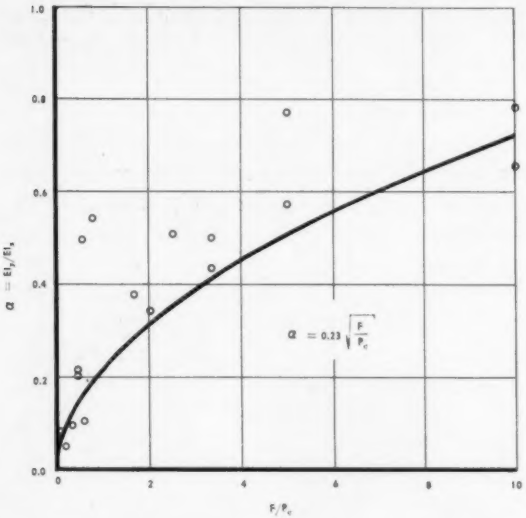


Fig. 14—Relation between α and F/P_c (center load)

lected; only the total post-tensioning force was considered, excluding that case where only bars over the supports are provided. Center and edge loading yielded different formulas, namely:

$$\text{Center load: } \alpha = 0.23\sqrt[2]{\frac{F}{P_c}}$$

$$\text{Edge load: } \alpha = 0.10\sqrt[3]{\frac{F}{P_e}}$$

The points resulting from our tests and the curves given by these formulas (chosen to be a lower boundary of the test points) are shown in Fig. 14 and 15.

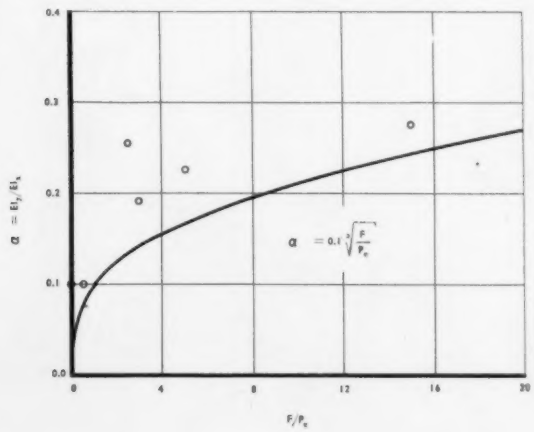


Fig. 15—Relation between α and F/P_e (edge load)

If a bridge is subjected to a load resulting from several trucks, P (center or edge) is not the total load, but only the portion concentrated at the peak of the moment coefficient curve. This means for most practical cases that the applied center load P_c is equal to two wheel loads, whereas the applied edge load P_e is usually equal to one wheel load.

Increase in moment coefficient due to slip and incomplete interaction of shear keys

It was felt that a suitable way of taking into account the effect of slip and incomplete interaction of the shear keys was to increase the value of the maximum moment coefficient. This increase (in percent) versus F/P was plotted as an upper boundary to the test points for the two cases—with and without shear keys (Fig. 16). Since the shear keys of our laboratory bridge were stronger than those found in practice, the results of a field test[1] on a bridge with normal shear keys were also included to find the lower curve of Fig. 16.

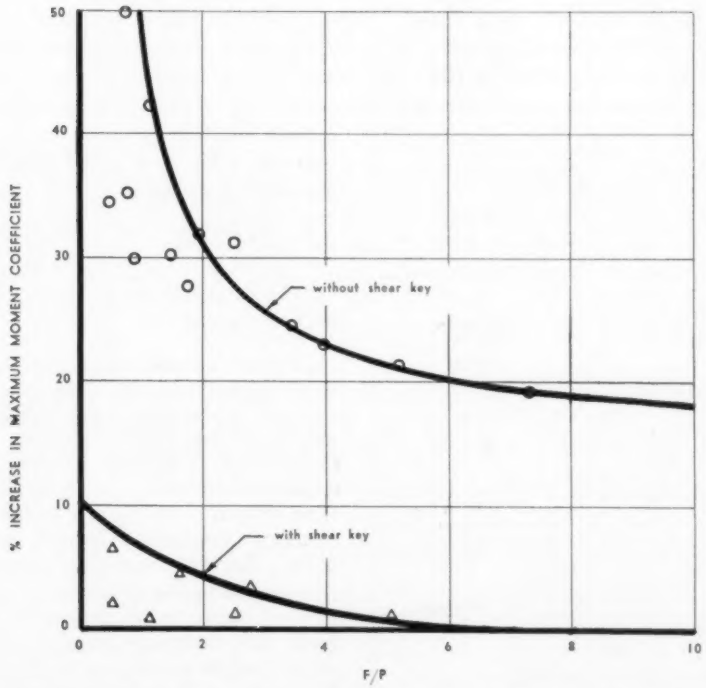


Fig. 16—Percentage increase in moment coefficient due to slip and incomplete interaction of shear key (center or edge load)

CONCLUSIONS AND RECOMMENDATIONS

1. The theory of orthotropic plates[2] was found applicable to the analysis of multibeam bridges. The correlation between the results of the theory and those of the tests was very close as long as only little displacement developed between adjacent beams. When the relative displacement between adjacent beams becomes decisive, the theory of orthotropic plates may still be used, but an empirical modification is necessary.

2. Design of a multibeam bridge is governed mainly by the longitudinal bending moments. These bending moments can be calculated by loading a single beam with a fraction of the wheel loads (equivalent loads) which produces the same moments as present in the bridge. This fraction is easily obtained from tables and curves.[2,3]

3. Two load conditions can be critical—a load concentration at either the center of the bridge or at midspan of an edge beam. The maximum S_M value (coefficient of longitudinal bending moment) for a given bridge is always higher for edge loading than for center loading, but since in the latter the possible load concentration is larger, the critical bending moment may occur in the center beam. Therefore, both load conditions have to be checked.

4. Shear can be critical either by punching through of a wheel load or in diagonal tension in an individual beam. The factor of safety against punching shear may be ascertained in the usual manner. The diagonal tension stresses may be calculated approximately by subjecting an individual beam to a certain fraction of wheel loads. It is safe to take the same fraction as derived for bending.

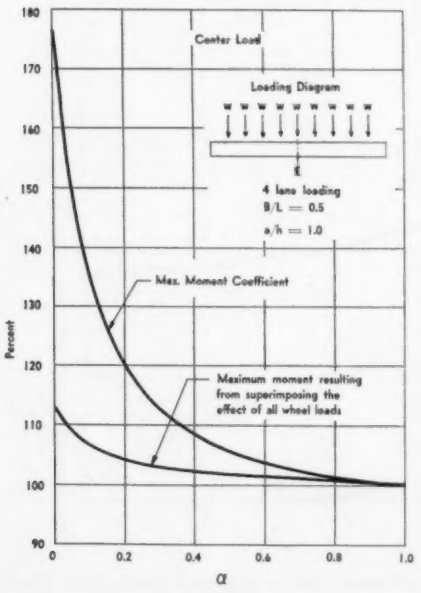


Fig. 17

5. No factor of safety should be required for uncertainties in the lateral load distribution as such for the following reason:

An error in estimating α produces a decisive change in the maximum moment coefficient. Fortunately, the maximum longitudinal bending moment resulting from a superposition of all load influences is less sensitive to errors in α. In Fig. 17 the maximum moment coefficient of a bridge and its maximum longitudinal bending moments under a four lane loading are plotted as a function of α, taking both moment and coefficient arbitrarily as 100 percent for $\alpha = 1$. While the maximum moment coefficient varies about 80 percent over the entire range of α (from 0 to 1), the maximum moment increases only by 12 percent (for edge loading only 8 percent).

6. The ultimate capacity of a prestressed multibeam bridge of normal construction can be expected to closely approach the sum of the capacities of all the members, provided that no punching shear failure occurs prematurely.

From Conclusions 4, 5, and 6 it follows that the safety of a multibeam bridge can be guaranteed if:

(a) The individual beams under the equivalent loads derived from the theory of orthotropic plates fulfill all the requirements of the specifications, and

(b) The punching shear stresses under a wheel load do not exceed the allowable limits.

7. The theoretical relationship between α (ratio of lateral to longitudinal bending stiffness) and β (coefficient of torsional rigidity) established by A. Roesli[2] was confirmed by our tests. For the calculation of α empirical formulas were derived from our tests as a function of F/P (total post-tensioning force divided by applied concentrated load). A distinction between center and edge loading must be made.

8. The influence of slip and incomplete interaction of the shear keys can be considered by increasing the maximum moment coefficient S_M as shown in Fig. 16.

9. Good lateral load distribution for a given bridge depends on the effectiveness of post-tensioning and shear transfer between adjacent beams. The predominant influence is that of strong shear keys. The question of when it is economical to apply post-tensioning can be judged from case to case by making use of the various charts given in this paper.

Post-tensioning

If post-tensioning is provided, care has to be taken to prevent the beams from cracking laterally. The beams are always a bit warped with respect to a vertical longitudinal plane, due to the unequal pretensioning of the longitudinal tendons. The lateral post-tensioning tends to straighten the beams; this can cause lateral cracks across the sides of the beams. To prevent such cracks grouted bearing areas should be provided between all beams at the location of the post-tensioning bars.

Among various combinations of post-tensioning, greatest efficiency is attained by arranging the bars either at the quarter points or by distributing them over the entire bridge. A relatively high efficiency may also be obtained by providing bars over the supports and at midspan, as might be more practical in the field.

ACKNOWLEDGMENTS

This investigation has been carried out in connection with research sponsored by the following: Pennsylvania Department of Highways; U. S. Bureau of Public Roads; Concrete Products of America, division of American-Marietta Co.; American Steel and Wire Division, United States Steel Corp.; John A. Roebling's Sons Corp.; Reinforced Concrete Research Council; and Lehigh University.

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Title No. 54-29

Proportioning, Control, and Field Practice for Lightweight Concrete*

By TRUMAN R. JONES, JR.† and HENSON K. STEPHENSON‡

SYNOPSIS

The most common problems encountered in batch proportioning, handling, placing, and finishing of structural quality concrete made with uncoated expanded shale and clay aggregates produced in Texas are discussed. Extensive laboratory studies have been made using expanded shale aggregates from three different sources and one expanded clay aggregate. Comments, conclusions, and recommendations are based upon observations made in the laboratory, on several major structures in the field, and at several commercial plants manufacturing lightweight aggregate concrete products.

A simple method is presented for the proportioning and control of the concrete batch. Practical solutions are given for certain of the field problems frequently experienced in handling, transporting, and stockpiling the aggregates. Practical suggestions for handling, placing, and finishing of the concrete are also discussed.

INTRODUCTION

There are certain characteristics peculiar to uncoated expanded shale, clay, and slate aggregates and lightweight aggregate concrete. When lightweight materials are handled in the conventional way, unnecessary problems are created in proportioning, handling, placing, and finishing that detract from the quality of the finished product and discourage use of the material. This paper describes and discusses these characteristics and suggests methods by which the problems may be eliminated. Due to the highly variable nature of the materials, no specific numerical data are given. Suggestions and conclusions are based upon observations of laboratory tests using three expanded shales and one expanded clay from different Texas producers, field uses on several major structures, and plant uses of several concrete products manufacturers. It is believed that the suggestions and conclusions apply in a qualitative way to the use of all uncoated expanded clay, shale, and slate aggregates.

GENERAL CONSIDERATIONS

Regardless of the *potential* any given proportion of concrete ingredients may have for developing desirable physical properties such as high tensile

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and compressive strengths, minimum shrinkage, and minimum creep under load, this potential will not normally be reached in the field unless the proportioning also affords such things as: simplicity in batching and handling; the proper workability for economical mixing, placing, and finishing without segregation; and uniform quality of the final product. Using lightweight aggregates these desirable ends are not all accomplished in the same manner as with the more familiar heavy aggregates. However, lightweight aggregate concrete having practically all of the desirable properties and lacking some of the undesirable properties inherent in heavy aggregate concrete can be produced with economy of time, energy, and money when the engineer and contractor thoroughly understand the materials.

Several characteristics peculiar to lightweight aggregates require special consideration. (1) The aggregates are sharp and angular throughout the range of sizes, even particles passing a No. 200 sieve, and this tends to make concrete harsh and unworkable just as crushed rock makes a concrete more harsh than sand and gravel. (2) The fines have a higher unit dry weight than the coarse aggregate and the method of manufacture produces an extremely dry material; both factors combine to make a product that segregates badly if not counteracted. (3) The aggregates are highly absorptive and the rate of absorption is variable, making it impractical to use specific gravity values in batch proportion computations and creating a problem in maintaining uniform workability on the job. (4) The aggregates are lighter than the concrete in which they are used, causing coarse aggregate to float to the surface when improperly handled. (5) The light weight of the concrete increases the normal concrete tendency to entrap air and form honeycomb.

WORKABILITY

A properly proportioned workable batch of lightweight concrete made with expanded clay, shale, or slate aggregates has much the same general appearance as a good workable batch of conventional weight concrete. It should have enough mortar to coat and carry the coarse aggregate during troweling without having an excess of fines. It should be sufficiently plastic for easy handling and sticky enough to hold together without cracking when being vibrated or rodded.

Particle shape and gradation

Workable mixes of sand and gravel concrete will usually have from 35 to 45 percent fines (passing the No. 4 sieve) and 65 to 55 percent coarse aggregate with a high percentage of rounded and sub-rounded particles which improve workability. The expanded shales and the expanded clay investigated were typical of many structural quality lightweight aggregates produced throughout the country, with a maximum size of $\frac{3}{4}$ in. The particle shape throughout the range of sizes is predominantly sharp and angular, even for the material passing the No. 200 sieve. These characteristics make it necessary to use

a higher percentage of fines to get the workability that is desired. When the fine and coarse fractions are graded in accordance with ASTM C 330-53T, a good lightweight mix will usually require from 45 to 60 percent fines and 55 to 40 percent coarse aggregate, in combination with other factors, to produce workability. Coarse and fine aggregate, meeting the ASTM gradation requirements individually, when blended in volume proportions varying from two parts coarse: one part fine to one coarse: two fine will usually give the necessary fines.

Using these proportions the desired workability can be obtained by lubrication which can be provided with cement paste, water, fine blow sand, air, or some other agent. Using extra cement paste to make the batch workable is unsatisfactory because the proportions required are such that it becomes too expensive and the shrinkage of the finished product is too great. Excessive water is undesirable since it reduces the strength, increases bleeding, increases segregation, and increases shrinkage. The use of blow sand is not particularly desirable because it rapidly increases the unit weight of the concrete and increases the shrinkage, but it does help raise the modulus of elasticity.

After studying some 50 different batch proportions using different aggregates and aggregate gradations, water contents, cement contents, various types and quantities of sand, and several commercial air-entraining agents, it is concluded that the most ideal lubricant for this type of concrete is entrained air. When properly used it greatly increases the workability and reduces the bleeding with apparently little if any adverse effect on the other properties. From the many tests conducted, it appears that in a well graded mix with normal cement factors, entrained air contents of 4 to 7 percent, as determined by ASTM C 231-54, with the added precaution that the specimen must be vibrated to eliminate entrapped air, produce good workability without reducing the ultimate strength appreciably. When the air content is increased above 7 percent the effect on strength becomes more noticeable, and for higher air contents the strength is reduced rather rapidly. Note that this air content is some 2 to 3 percent higher than the amount normally used with sand and gravel. The amount of air entrained is affected by the cement content, the water content, the aggregate proportions, particle shape, and the temperature of the batch, but usually the proper amount of air can be entrained by using from $\frac{3}{4}$ to $1\frac{1}{4}$ oz neutralized Vinsol resin per sack of cement. It is important that the Vinsol resin be thoroughly dispersed in the mixing water before introducing cement and aggregate into the batch. Work by other investigators shows that entrained air in concrete improves the wearing quality, decreases permeability, and increases the resistance to freezing and thawing, all of which may be considered as bonus benefits in the case of lightweight concrete, since the air is entrained primarily for workability.

Absorption

The capacity of the aggregates to absorb water varies considerably, as does the rate of absorption, but all of the uncoated expanded clay, shale, and

slate lightweight aggregates are capable of holding rather large quantities of water. The coarse aggregate will absorb water equal to 10 to 25 percent of its dry weight and the fines will absorb 15 to 30 percent if allowed to soak for as long as an hour. Water absorbed by the aggregate is evaporable, but it apparently has no effect on the workability of the concrete batch nor does it need to be considered in computing the water-cement ratio when estimating concrete strength. The only water in the batch that seems to affect strength and workability is the "free" water that is not absorbed by the aggregate. Aggregates introduced dry into the mixer may produce a concrete that is workable when discharged, but before the concrete is placed and consolidated it is so stiff that it can not be handled.

The problems of loss of workability due to absorption, segregation of aggregate, and loss of fines can be corrected satisfactorily by prewetting the aggregates before shipping and at least 24 hr before batching. It is recommended for all jobs requiring good quality concrete that the aggregate be wetted at the source of supply while on conveyor belts or while being loaded by using a fine spray or some other satisfactory device supplying measured quantities of water. It is desirable that from 8 to 15 percent water, by weight, be added at this time; the optimum amount for a given source and gradation is fairly constant. Care should be taken that the velocity of the water does not cause segregation and that it is not added more rapidly than the aggregate can absorb it. At least one manufacturer in Texas has installed a short screw type conveyor with the belt system and water is added at this point, producing a uniformly wet aggregate without segregation.

Proper amount of prewetting will keep segregation of aggregate and loss of fines to a minimum and still allow the materials to flow freely enough for easy handling. It will also allow greater variation in mixing and handling time of concrete without appreciable changes in the workability, since it reduces the rate of absorption of mixing water by the aggregate. Concrete made with aggregate that has not been prewetted properly frequently settles or shrinks in the forms due to the absorption of the mixing water by the aggregate. A fairly uniform water content can be maintained in stockpiled prewetted aggregate by sprinkling the surface of the stockpile to offset effects of evaporation. The only apparent disadvantages of prewetting at the source of supply are that the fines may bulk from 0 to 35 percent when wet to various degrees, and the freight bill will be greater for a given dry quantity of aggregate when shipped wet. Since the material when dry cannot be shipped or handled without severe segregation and loss of fines and cannot be batched with uniform workability and quality, it is recommended that prewetting at the source of supply be *required* on all jobs.

Due to the lighter weight of the materials involved, lightweight concrete of a given workability does not slump as much as sand and gravel concrete. Lightweight concrete with 4 to 7 percent entrained air (as determined by ASTM C 231-54 with added precaution that the specimen must be vibrated to eliminate entrapped air) and having a slump of 2 to 3 in. can be placed in forms,

around steel, under conditions that would require a slump of 3 to 5 in. for normal sand and gravel concrete. Actually slump is a poor measure of workability and should only be used to indicate a uniform consistency for particular batch proportions.

BATCH PROPORTIONING

The absolute specific gravity of expanded lightweight aggregate has no relationship to the volume of aggregate since most of the particles have a large number of fully enclosed air voids. Absorption characteristics of the aggregates are such that the standard test methods do not give consistent results for the apparent specific gravity or for the bulk specific gravity. As a matter of fact the standard test for determining absorption of fine aggregates does not even give a consistent indication of the saturated surface-dry condition. In addition to the problems of determining specific gravities, the unit weight of the fines is considerably higher than the unit weight of the coarse aggregates. When all of these factors are considered, it becomes apparent that a batch cannot be proportioned on an absolute volume basis, or any of the other more or less common methods used with heavy aggregates.

The simplest method of arriving at a satisfactory mix appears to be on a dry loose volume basis with trial and error procedures to determine the yield, etc. All Texas suppliers of expanded clay and shale aggregates can furnish coarse and fine aggregate graded in accordance with ASTM C 330-53T and when gradation falls within limits such as this, the dry unit weights from a given source are rather consistent. Thus, in actual field practice, the concrete can be batched by weight or volume after basic mix proportions have been determined.

The lightweight aggregates tested produce concrete with ultimate strengths equal to or greater than strengths normally expected using sand and gravel with the same cement factor and free mixing water content. This strength can be obtained with quite a wide variation in gradation; for instance, the change in strength with the proportions of aggregate varying from two parts coarse: one part fine to one coarse: two fine is not great so long as the cement factor per cu yd is maintained and the slump is held constant. By varying the gradation and entraining air it is possible to produce concrete with a dry weight of 90 to 100 lb per cu ft having a strength of 5000 psi or more with reasonable cement factors. Adequate strength can be attained with greater fine aggregate proportions than those given here, but excessive fines increase shrinkage and density of concrete and it is desirable to keep fines to a minimum consistent with good gradation and workability.

A recommended procedure for arriving at mix proportions is as follows:

1. Assume a basic design satisfying the requirements for strength, durability, and workability such as: two parts coarse aggregate; one part fine; 5 bags cement per cu yd; 3 in. slump; 5 percent air content. (When properly mixed and cured 7 days, the expected

28-day compressive strength will be 3500 to 5000 psi, depending upon the properties of the aggregate and the properties of the cement.*)

2. Place from ½ to ⅔ the estimated amount of mixing water needed, containing the neutralized Vinsol resin, in the mixer. Measure the amount of water. (When the aggregate is dry the total amount of water needed may be from 10 to 15 gal. per bag of cement for 5- and 6-bag batches.)

3. Measure quantities of coarse and fine aggregate in accordance with the assumed mix design, such as 2 cu ft coarse and 1 cu ft fine; weigh these quantities and place in the mixer. If the aggregate is dry, allow mixer to run 15 or 20 min so that the absorption rate of aggregate will level off and not affect the concrete slump too greatly. If the aggregate is wet make certain that the quantity measured takes into account the change in volume due to prewetting, and in either case take a small sample of the aggregate for a moisture determination.

4. Assume a concrete yield of 70 to 80 percent of the total aggregate used and determine the amount of cement to be added. Add this cement to the mixer. In this example the estimated yield would be 80 percent of 3 cu ft or 2.4 cu ft. The total cement per cubic yard = $5 \times 94 = 470$ lb, and the cement for this batch = $(2.4/27)(470) = 41.8$ lb.

5. Add sufficient additional water to give the required slump. (Measure the water.)

6. Determine the air content (ASTM C 231-54, plus vibrating the specimen).

7. Determine the total weight of concrete in the batch and the weight per cu ft (ASTM C 138-44, plus vibrating the specimen). Divide the total weight by the weight per cu ft to determine volume of concrete produced, using the measured quantities for this first trial batch.

8. Check the amount of cement used against the amount of cement required for this quantity of concrete and the air content obtained against that desired for this batch.

9. Correct the quantities and repeat all steps until the desired batch is obtained.

The total water used is the amount in the aggregate plus the total amount added in Steps 2 and 5 above and should be recorded as the total amount of water in the basic mix. If the quantity of cement added in Step 4 is less than the amount needed, the correct batch can usually be obtained very quickly by adding cement to the batch in the mixer. An experienced laboratory man will usually get the desired proportions on the second or third trial.

FIELD PRACTICE

Batching

When batching by weight, the weight of the water in prewetted aggregate should be added to the weight of dry aggregate required in the batch for setting the scale weights and should be subtracted from the total amount of water required in the batch to determine the additional amount of water to be added. When batching by volume the bulking of the prewetted fines must be taken into account in measuring the aggregates, and water in the aggregate must be deducted from total water required in the basic mix to determine the amount to be added. After the mixer has been charged with the first batch on any

*For aggregates with properties similar to those under study, reducing the slump 1 in. by varying the water content increases ultimate compressive strength approximately the same as adding 1 bag of cement per cu yd while holding the slump constant; likewise increasing the slump decreases the strength. This is not a basic principle but merely an observation of the performance characteristics of a particular kind of concrete. Apparently reducing the water content enough to reduce slump 1 in. lowers the water-cement ratio approximately as much as it would be lowered by adding a bag of cement.

given job it should be mixed for the required time, the air content should be determined, and a slump test should be made. When the air content is correct, the actual slump should be compared to the design slump and the mixing water adjusted to compensate for any differences. Measuring concrete consistency by the slump test gives a very good indication of the free mixing water in a given batch design and serves as a good field check for controlling the water, when conducted by an experienced man. The care and technique in making a slump test frequently affects the measured result more than the properties of the concrete.

Aggregate for any given placement of concrete should have a uniform moisture content. Otherwise the quantity of water to be added will have to be adjusted for every batch. Of course this is true with any kind of concrete but the difference in water from dry to saturated condition is much more significant for lightweight aggregates than for heavy aggregates.

Mixing

Thorough mixing of the concrete batch is imperative in producing quality concrete. This is especially true with lightweight aggregates and the average mixing time for good results is somewhat longer than for sand-gravel concrete. Due to the big difference in unit weight between cement paste and aggregate, it is necessary to run conventional mixers at slightly higher speeds than with sand and gravel to get best results. Efficiency of different types of mixers varies, and this should be taken into account when specifying mixing time. In writing specifications a statement could be made to the effect that the mixing time will be established by the engineer for the basic mix proportions and specific equipment to be used; but in no case shall it be less than some minimum such as 3 min.

Placing

Forms for lightweight concrete can be designed for much lighter loads than is customary for sand-gravel concrete. Wet weight will seldom run over 115 lb per cu ft and most generally will vary between 100 and 110 lb per cu ft for concrete testing about 5000 psi, and will be less than 100 lb per cu ft for concrete testing 3000 psi.

The lighter weight of materials increases the tendency of concrete to honeycomb and form voids due to entrapped air. The exposed surfaces often show many small pock marks or air bubbles, and when these are visible on the surface the interior concrete very likely contains a considerable amount of entrapped air also. These defects can be eliminated by proper vibration.

The importance of proper vibration cannot be overemphasized. Strength of properly vibrated lightweight concrete may be 50 percent or more higher than for the same mix rodded in the usual manner. It is recommended that high frequency vibration be *required* on all lightweight concrete jobs. Best results have been obtained by vibrating at a frequency of approximately 7200 rpm—about twice the frequency most commonly used for sand and gravel concrete. This higher frequency is very effective in consolidating the concrete

and eliminating honeycomb and surface blemishes. Lower frequencies can be effective if the equipment available can exert vibrations of fairly large magnitude. For thin sections, form vibrators are adequate but for thick sections or mass concrete, internal vibration will also be necessary. The length of time that the vibration is applied must be determined by the engineer on the job, since excessive vibration segregates the concrete, floating large aggregate to the surface with the cement and fines going to the bottom, whereas insufficient vibration leaves the concrete full of trapped air voids. A well proportioned batch of lightweight concrete, using prewetted aggregates, does not segregate as badly as heavy concrete but segregation is much more discernible in the lightweight material at the time it is occurring due to the floating of the coarse aggregate.

Finishing

Excessive floating or troweling works large aggregate to the surface just as excessive vibration does. This leaves a rough surface with the large aggregate protruding and further troweling only aggravates the condition. On flat work such as slabs requiring a smooth finish it is recommended that the coarse aggregate be moved away from the surface before floating and troweling operations are undertaken.

One satisfactory procedure is to strike off the surface and bring it to grade with a high frequency vibrating screed. This leaves sufficient mortar on the surface to give a good finish. The vibrating screed is particularly adaptable to plant operations and to formed slab work where a number of placements of uniform width are to be made. If the screed can be made adjustable in length the requirement of uniform width is eliminated.

Another very satisfactory method is the use of what is called a "jitterbug" in some localities. This is a tamping device with the contact surface consisting of wire mesh normally ¼ in. square. When the concrete is tamped the large particles are moved away from the surface and only the material finer than ¼ in. remains for the floating and troweling operation. The concrete surface should be brought to proper grade before "jitterbugging." If the finish is too coarse, the mesh size can be reduced.

Best results are obtained when the surface is finished with a minimum number of floating and troweling operations.

SUMMARY

1. Mix proportions should be set up with the aggregates on a dry, loose volume basis; proportions can best be determined by trial batches.

2. Basic mix proportions can be converted to weights for batching in the field.

3. A much higher percentage of fines is required than for conventional sand and gravel concrete.

4. Neutralized Vinsol resin or other approved air-entraining admixture should be used to entrain sufficient air for good workability; this appears to

be from 4 to 7 percent (as determined by ASTM C 231-54 with the added precaution that the specimen must be vibrated to eliminate entrapped air).

5. Aggregate should be prewetted before handling or shipping to prevent segregation and should be prewetted at least 24 hr before batching to help control the absorption and give a uniformly workable mix. The water content of the aggregate should be uniform for any given placement.

6. Special care should be taken to assure thorough mixing. Mixing time is longer than that common for sand and gravel concrete; the mixer speed should be faster.

7. Mixing water content in the field can be maintained at a constant level by checking the consistency of the concrete with the slump cone test. Constant values of slump indicate constant values of free mixing water for specific batch proportions.

8. Expanded shale and clay concrete can be placed and finished with approximately 2 in. less slump than is required for a given situation using sand and gravel.

9. Vibration is necessary for good work. Frequencies around 7200 rpm give excellent results; this is approximately twice the frequency most commonly used for sand and gravel concrete.

10. Flat surfaces will usually require the use of a "jitterbug" or high frequency vibrating screed to maintain sufficient fines on the surface for finishing. Floating and troweling should be kept to the minimum necessary for a smooth surface.

11. Forms can be designed for much lighter loads.

Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1958, for publication in the June 1958 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Langstone bridge, Hampshire, England

W. A. EYRE, *Ingeniøren* (Copenhagen), B Edition, V. 66, No. 19, June 1957, pp. 475-482
Reviewed by JESPER STRANDGAARD

Describes construction of the 958 ft long bridge consisting of 29 spans 32 ft each and four shorter spans designed as bents to carry the longitudinal forces. The 22-ft roadway is carried by 16 precast, prestressed 18 in. square concrete beams, each reinforced with two 1⅛-in. Macalloy rods. Freyssinet cables were used laterally. Pile bents of five 16 in. sq precast concrete piles, up to 80 ft long, with a cast-in-place cap, are used as piers.

Overpassing a two-track railroad with a prestressed skewed (34 deg) slab (in German)

O. BERR and R. MARX, *Der Bauingenieur* (Berlin), V. 31, No. 10, Oct. 1956, pp. 361-372
Reviewed by ARON L. MIRSKY

Previous bridge, a war casualty, had been built normal to center line of railroad by virtue of a reverse curve in the road (German Federal Highway 51); result, numerous accidents. New bridge alignment was dictated by highway layout, and result is a very sharp skew (34 deg from center line of highway to center line of railroad). Problems of clearance and height made bridge thickness an important factor in selection of type of structure. Choice was a prestressed slab. A study of the method of prestressing to be used was made necessary by the shape (acute corners) of the slab; a model analysis was also made (for description, see article by Vogt in *Beton-und Stahlbetonbau*, V. 48, 1953, pp. 281-285). Current paper, besides discussing these points, covers design, analysis, and construction in some detail.

Construction

Tunnel linings

O. DAWSON, *Bulletin*, Institution of Civil Engineers of Ireland (Dublin), V. 83, No. 1 (Session 1956-57), Dec. 1956, pp. 17-33

Summarizes types of tunnel linings in use and available including several newly developed precast concrete segmental linings; describes the advantages and disadvantages of each together with construction methods they necessitate. Concludes with five pages of illustrations and ten literature references, principally British.

Projection planetarium and shell construction

WALTHER BAUERSFELD, *The Engineer* (London), V. 203, No. 5286, May 17, 1957, pp. 755-757; No. 5287, May 24, 1957, pp. 796-797
Reviewed by ARON L. MIRSKY

Planetaria require two essential items: a projection apparatus and a hemispherical ceiling or dome on which to project the images of the stars. Development of first Zeiss planetarium, that for Munich in 1923, involved development of ferro-concrete for thin shells; this in turn led to the development of cylindrical shells (Dischinger, Finsterwalder) and thence to true thin shells. It all makes a fascinating story; one does not have to be an astronomer (amateur or professional) or mechanical engineer to enjoy the first part of this James Clayton Lecture on the projector, or a structural engineer to appreciate the second, on shells.

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 6, Dec. 1957, *Proceedings* V. 54. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

Construction Techniques

Effect of the steam curing diagram on cured concrete depending on the quality of cement (in Hungarian)

Gy. Kunszt, *Építöanyag*, V. 8, No. 4, 1956, pp. 146-154

Hungarian Technical Abstracts V. 9, No. 2, 1957

The effect of 12 types of curing on concretes made with portland, trass-portland, and blast-furnace slag cements was investigated. It was found that the curing effect is dependent to a great extent upon the shape of the curing diagram. A definite relationship has been established between reduced time, reduced temperature, and strength properties. This relationship was found to vary for the different kinds of cement used.

Vibration of concrete

Joint Committee Report, The Institution of Civil Engineers and The Institution of Structural Engineers, London, 1956, 65 pp., 8 s.

Comprehensive committee report on the consolidation of concrete. Discusses specification of concrete materials for concrete to be vibrated; forms to contain vibrated concrete; the various types of vibrators available: immersion, clamp-on, surface, table, and miscellaneous. Presents a section on recommended practice for the use of vibration; recommendations for further research and suggested specification additions for concrete to be vibrated. Includes a number of comparisons of vibrated and non-vibrated concrete representing results of considerable research.

Steam curing of concrete and its optimal conditions (in German)

S. Bleier, *Acta Technica Academiae Scientiarum Hungaricae* (Budapest), V. 14, No. 1-2, 1956, pp. 39-54

Hungarian Technical Abstracts V. 9, No. 1, 1957

The essence of the steam treatment applied for aiding and accelerating the setting of concrete is the increase in volume of the green concrete due to the action of heat. The increase in volume allows the optimal development of the increasing compactness of the cement brought about by the chemical transformation permitting rapid setting at the maximum strength conforming to the composition of the concrete. For optimal effect curing time should be split into two phases: the heating period and the continuous action of heat. During heating the internal pressure developed in the concrete should always stay below the ambient pressure. During the reduction of pressure and cooling subsequent to steaming, the internal pressure in the concrete should remain permanently above the ambient pressure; 12 atmospheres may be regarded as the optimum. Steaming at optimum pressure is the most dependable means of establishing preliminarily the strength of concrete. The optimum duration for outdoor steam curing at atmospheric pressure is 3 hr, maximum 4 hr, for bodies of 20 cm thickness.

Note on the "revibration" of concrete (Note sur la "revibration" des bétons)

R. Peltier, *Annales des Ponts et Chaussees* (Paris), V. 126, No. 6, Nov.-Dec. 1956, pp. 811-819

Reviewed by Aron L. Mirsky

Three mixes, ranging from dry to wet and corresponding to those usually used in French construction, were tested using various periods of vibration and of rest between vibrations. Results verified those obtained by Dutron in Belgium: wet concretes are not harmed by revibration, so long as setting of the concrete has not commenced, but such revibration is very harmful to dry concrete, even if the time between the two periods of vibration is very short.

Colloidal concrete (Kolloidaler Beton)

G. Brux, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 1, Jan. 1957, pp. 18-22

Author's Summary

The Colcrete process, which was developed in England 20 years ago, is discussed. Coarse aggregate over 40 mm in size is placed in the formwork, and the voids are then filled with colloidal colgrout. Examples of structures built by means of this process are given, e.g., retaining walls, heavy foundations, and slabs carrying traffic loads.

Industrial practice in high turbulence mixing (La practique industrielle du malaxage a haute turbulence)

G. Leenhardt, *Revue des Materiaux* (Paris), No. 498, Mar. 1957, pp. 79-82

Reviewed by Phillip L. Melville

Examples of use of mortar prepared in a high turbulence mixer are given. Results of grouting on three jobs are reviewed.

Depth of penetration of concrete into wood-wool sheets for light construction (Eindringtiefe von Beton in Holzwolle-Leichtbauplatten)

KURT GAEDE, *Bulletin* No. 126, Deutscher Ausschuss für Stahlbeton (Berlin), 1957, pp. 19-31, 10 DM (for entire bulletin)

Reviewed by ARON L. MIRSKY

Sheets of wood wool cemented together with portland cement (sometimes gypsum) have been used as form lining. Question arose whether penetration of cement into these liners left cast-in-place concrete porous, leaving reinforcement in danger of corrosion. Results of tests on 50 mm sheets with 10 cm of concrete, of mixes ranging from wet to dry, showed that while mortar did penetrate some distance, particularly with the wetter mixes, amount of mortar penetrating sheets was small, and it is not necessary to protect surface of the sheet before concreting; however, for thermal calculations, a 2 to 5 mm depth of the wood-wool sheet should be assumed filled with mortar. Reinforcement should be supported at the desired level by chunks of cured mortar (chairs).

An expression is also derived for calculating depth of penetration of mortar, and a comparison of measured and calculated values is given.

Design

Graphic solution of the slab problem (La solution geometrique du probleme de la dalle—La methode des points Emmy)

J. F. BUCHI, *Revue Technique du Batiment* (Paris), No. 37, Jan. 1957; No. 39, Mar. 1957; No. 44, Sept. 1957. *Il Cemento* (Milan), No. 8, 1957

AUTHOR'S SUMMARY

A "layman's" method combining precision, simplicity, and understandability. It is recommended for practical purposes as advanced mathematics are replaced by a visual method. The first eight parts of the paper deal with external stresses while the ninth, and last, treats internal stresses.

Dealing with external stresses the author reveals tests and practical conclusions on such subjects as: uniformly loaded rectangular floor slabs; correlation of the Marens method with the *Points Emmy* (graphic) method; hydrostatic loads (flat slabs supported on four sides and flat slabs supported on three sides); slab supported on four sides with concentrated load; correlation of loads applied to one slab; and flat plate type of flat slab.

Part 9, dealing with internal stresses, covers methods of correlation to simplify the use of classic theory in reinforced concrete.

Materials

Pressure grouting fine fissures

Technical Report No. 6-437, Waterways Experiment Station, Vicksburg, Miss., Oct. 1956, 75 pp., $0.75

An investigation to determine the degree to which penetration of fine fissures by grout is influenced by surface texture of fissure, pumping pressure, water-cement ratio, chemical fluidifiers, and finely divided mineral additives, as well as the effect of these factors upon the quality of the hardened grout films. The investigation also included the effect of patented materials for use as admixtures in grouting. Principal conclusions are that the ratio of crack width to be grouted to grain size should probably be three or more; that any material which will reduce bleeding of the grout and increase setting time will improve results. Materials found to improve results were such patented materials as Intrusion Aid and RDA, and finely ground mineral admixtures such as granulated blast-furnace slag, pumicite, and opaline shale.

On the contraction of hardening cement (Über die Schrumpfung des erhaertenden Zementes)

W. CZERNIN, *Zement-Kalk-Gips* (Wiesbaden), V. 9, No. 12, Dec. 1956, pp. 525-530

Reviewed by H. H. WERNER

Self-desiccation in mortars and concrete is reviewed and a relationship between the volumenometric measurements of contraction, the chemically fixed water, the compressive strength, and the contraction at low tenperature are investigated.

Contribution to the study of standard mortar (Contribution a l'etude du mortier normal)

MICHEL VENUAT, *Revue des Materiaux* (Paris), No. 493, Oct. 1956, pp. 221, 233; No. 494, Nov. 1956, pp. 255-271; No. 495, Dec. 1956, pp. 283-293

Reviewed by PHILLIP L. MELVILLE

A thorough study of mortars used in cement testing. After a historical introduction, the requirements for an international standard are given. Need for improvement and even-

tual replacement of the French standard is noted. The effect of sundry sands, proportions of sand, cement, and water, mixing methods, and types of apparatus were investigated. It is concluded that testing of mortar is more informative than testing of cement paste; the maximum grain size should be 2 mm; the sand:cement ratio should be 3:1; the water-cement ratio should be 0.50; the specimens should be 4 x 4 x 16 cm (for flexure, compression, and shrinkage tests); the mixing and placing should follow the Swiss procedure; curing should be in water at 20 ± C.

Terlite—Lightweight aggregate

JOHN GRINDROD, *Pit and Quarry*, V. 50, No. 3, Sept. 1957, pp. 132-134

Describes a full-scale research plant for the conversion of fly ash to lightweight concrete aggregate operated by the Central Electricity Authority, London, England. Discusses properties of resulting aggregate which is composed of sintered pellets of fly ash. Sintering is done in a sintering kiln fired by the carbon in the fly ash itself. Structural grade concrete (3000 psi and better) is possible at about 100 lb per cu ft using this new lightweight aggregate.

SO_3-content of aggregates: Extended investigations (SO_3-Gehalt der Zuschlagstoffe: Langzeitversuche)

KURT GAEDE, *Bulletin* No. 126, Deutscher Ausschuss für Stahlbeton (Berlin), 1957, pp. 1-18, 10 DM (for entire bulletin)

Reviewed by ARON L. MIRSKY

This sequel to *Bulletin* No. 109 ("Current Reviews," ACI JOURNAL, June 1953, p. 979) reports results of tests at ages of 26 and 104 weeks (previous report was based on tests at 6 weeks). Eighty-eight series were made and tested, using three cements (one portland, two blast-furnace) and one aggregate, but two basic types of mix ("heavy" or dense concrete, and "one-size" or no-fines, porous concrete), and two types of sulfates (gypsum and magnesium sulfate). Loss of strength was slightly lower for the porous than the dense concrete, was much greater for magnesium sulfate than for gypsum, and was higher for portland than for blast-furnace cement; but in all cases the loss occurred during the first 26 weeks, after which no significant change in strength was noted. Interestingly, the reduction in strength occurs in a narrow range of sulfate content, beginning at 1.2 to 1.5 percent and reaching a maximum at 2.4 to 3.0 percent (calculated as percentage of weight of cement plus 0.14 weight of aggregate).

Cement mixing to accelerate set; effect on shrinkage and swelling (Melanges de ciments pour accelerer la prise, leurs consequences sur le retrait et le conflement)

LOUIS FERET and MICHEL VENAUT, *Revue des Materiaux* (Paris), No. 496, Jan. 1957, pp. 1-11

Reviewed by PHILLIP L. MELVILLE

Mixtures of alumina with portland cement and alumina with high early strength cement were investigated to study swelling and shrinkage. Time of set was varied with the mixes from very slow to almost instantaneous. Strengths were found to decrease with time of set, durability became unreliable, shrink and swell increased with alumina contents. Mixtures of portland with high-early cement were less critical.

Selection of natural aggregates for various types of concrete work

J. D. MCINTOSH, *The Reinforced Concrete Review* (London), V. 4, No. 5, Mar. 1957, pp. 281-305

Describes the most important effects upon the quality of concrete produced by various types of aggregates. Suggests that when the effect of aggregate is due to an inherent property it may be necessary for special concreting purposes to select particular aggregates, but that many properties of aggregates can be controlled by the producer or provided for by changes in the proportioning of concrete.

Building with fly-ash: Aggregate plant at Battersea Power Station

Engineering (London), V. 183, No. 4761, June 7, 1957, p. 721

Reviewed by ARON L. MIRSKY

Fly ash from the power plant precipitator is dampened and formed into nodules which are then sintered. Resultant pellets are used as aggregates in concrete blocks, which possess light weight, high strength, and high thermal and acoustical insulating properties.*

*Cf. *Concrete and Constructional Engineering*, July 1957, pp. 255-256.

Quantitative determination of potassium with sodium tetraphenyloborate (Die quantitative Bestimmung von Kalium mit Natriumtetraphenyloborat)

W. Lieber, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 2, Feb. 1957, pp. 61-62

Author's Summary

With the potassium reagent *Kalignost*, sodium tetraphenyloborate, the determination of potassium can be carried out with a high degree of accuracy and without having recourse to test solutions. The method represents a combination of precipitation analysis followed by titration. Working instructions and some analysis results are given.

An improved procedure for the complexometric determination of calcium and magnesium (Eine verbesserte komplexometrische Calcium- und Magnesiumbestimmung)

H. Otterbein, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 2, Feb. 1957, pp. 57-60

Author's Summary

The complexometric determination of calcium and magnesium is improved by the fact that the end point is indicated by optical and electrical means with the aid of a photoelectric cell. The requisite apparatus is described and working rules are given. The accuracy for CaO is approximately 0.05 percent of the weighed portion for analysis, and for MgO it is approximately 0.1 percent.

Furthermore a set of working rules is given for a complete process of cement analysis with complexometric determination of CaO, MgO and free lime and nephelometric determination of SO_3. The analysis takes 2½ hr in the case of one sample, and 4 hr if two samples are dealt with at the same time.

Setting of portland cement (Das Erstarren des Portlandzementes)

R. Hayden, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 1, Jan. 1957, pp. 16-18

Author's Summary

From the occurrence of only small amounts of new compounds at the final set, the decomposition in alcohol of cement that has set, and the fact that it is thereby possible to recover the cement with its original setting properties, the theory is derived that setting is fundamentally attributable to capillary forces, which, in consequence of water absorption, increase in magnitude with the increasing surface area of the new compounds that are formed along with hydrate of lime.

Setting is an independent process. Interference with it does not affect the actual hardening process.

Conclusions in regard to slag clinker (Conclusions au sujet du clinker metallurgique)

R. Feron, *Revue des Materiaux* (Paris), No. 497, Feb. 1957, pp. 52-54

Reviewed by Phillip L. Melville

A brief discussion on the value of adding slag to clinker as far as cementing power is concerned. It is pointed out that storing clinker results in a reduction of free lime and changes tricalcic into bicalcic silicate with a loss in cementing power. The conclusions are as follows: slag cement is most economical and efficient with 15 to 20 percent slag; higher fineness will compensate for any weakness in the clinker.

Determination of the modulus of elasticity on 4 x 1 x 16-cm mortar prisms (Bestimmung des Elastizitäts-Moduls an Mörtelprismen 4 x 1 x 16-cm)

H. Pajenkamp, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 2, Feb. 1957, pp. 63-66

Reviewed by H. H. Werner

Modulus of elasticity and how it is affected by various factors is investigated for 4 x 1 x 16-cm mortar prisms. Good agreement between laboratory and field tests is obtained, affording a convenient means of determining the relative effect of influencing factors on the elastic properties.

Pavements

Significance of tests for highway materials—basic tests

Progress Report of the Committee on Significance of Tests for Highway Materials of the Highway Division, *Proceedings*, ASCE, V. 83, HW4, Sept. 1957, pp. 1385-1 to 1385-37

A summary of the common tests on highway materials including soils, asphalts, and concrete. Discussion of typical results of these tests and their significance. The report presents no new material but brings together data on all basic tests and their uses. Of

value for engineers now serving in responsible positions under the highway expansion program.

Convention hears report on nature of frost action

E. PENNER, *Heavy Construction News* (Toronto), V. 1, No. 27, Oct. 1957, pp. 6, 7, 10, 11

A paper presented at the 38th convention of the Canadian Good Roads Association dealing with the frost-heave process. Reports on research studies into the mechanism of frost heave and conditions necessary for its occurrence. Concludes with recommendations for the solution of the problem of frost action.

Precast Concrete

Development of Danish precast concrete

R. A. LARSEN and P. E. MALMSTROM, *Ingeniøren* (Copenhagen), International Edition, V. 1, No. 1, July 1957, pp. 2-12

Reviewed by JESPER STRANDGAARD

A brief account is given of the background for the rapidly increasing use of precast concrete elements in post-war building construction in Denmark. Through the necessary revisions of building codes and laws of financing the development is followed from the modest beginning with smaller structural elements to the present highly industrialized construction, where complete room units, floors and load-carrying walls are precast. The use of sandwich-type wall elements, where both the interior and exterior finish, and the insulation, make up the prefabricated unit, is described.

Prestressed Concrete

Concrete industries yearbook. Part 3 —Prestressed concrete

R. L. PECK, editor; preprinted by Pit and Quarry Publications, Inc., Chicago, 1957, 70 pp.

Complete discussion of prestressed concrete from the viewpoint of a manufacturer of prestressed concrete products. The discussions are not technical as regards theory or design but do include practical problems of production. Contents include: short sections on the history, theory, and design practice with a list of recommended consulting structural engineers; discussions of materials used and manufacturing equipment available with names of suppliers; various manufacturing processes which are in successful use; advantages and disadvantages, as well as technical features of the various products now available; examples of noteworthy and common applications of these products throughout the country. Concludes with a bibliography.

The 70 pages include numerous photographs of actual plants, products, and equipment. This publication is highly recommended for manufacturers and prospective manufacturers to provide both background on the product and its development as well as an up-to-date report on the growth of the industry, policies of competitors, and latest technical developments.

Properties of Concrete

Free CaO of cement and the effect of the admixture of Rajasil on the chemical resistance of cement (Das freie CaO des Zements und die Wirkung des Betonzusatzmittels Rajasil auf die chemische Widerstandsfähigkeit des Zements)

F. W. LOCHER, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 6, June 1957, pp. 221-231

Reviewed by H. H. WERNER

The theory by Meier-Grolman, regarding the resistance of concrete to aggressive agents, is investigated. Microphotographs of hardened cement paste show that not all Ca $(OH)_2$ is absorbed by the gel. The comparison of the maximum content of free CaO and C_3A with the mean values for the sulfate resistance is considered inconclusive.

In the second part of the paper, laboratory test with the admixture caused an increase in efflorescence and sulfate expansion and produced only an insignificant increase in sulfate resistance.

Corrosion of steel in a reinforced concrete bridge

R. F. STRATFULL, *Corrosion*, V. 13, No. 3, Mar. 1957, pp. 43-48

HIGHWAY RESEARCH ABSTRACTS June 1957

Studies to determine the cause of accelerated corrosion of reinforcing steel in a concrete bridge in a salt water environment showed that macro-galvanic corrosion was involved.

Resistivity and corrosion cell ratio charts indicate that the resistivity of concrete decreases as the deterioration of the concrete increases. The lower resistivity measurements of the concrete in the anodic areas indicated that the anodic areas could have been created by the following factors: (1) a greater moisture content in the anodic areas, and (2) a greater sea salt content in the anodic areas.

Cathodic protection may offer a means of curbing corrosion but certain precautions should be taken.

Structural Research

Photogrammetric measurements of deformations of structures

C. Moser and W. R. Schriever, *Research Paper* No. 30, Division of Building Research, National Research Council of Canada, Ottawa, Jan. 1957, 20 pp., $0.25

BRI Abstracts
First Quarter, 1957

One of the many non-topographical applications of photogrammetry is the determination of deformations of structures. Two methods are discussed, the single camera method and the two camera method. In most practical cases where only deflections in one plane are to be measured, the single camera method can be used. If deformations in three dimensions are required, the two camera method must be applied. Advantages and disadvantages of these methods and the accuracy of photogrammetric measurements are discussed. Examples of application include wooden beams, concrete beams, wooden roof trusses and railway track under load.

General

Specifications and costs. Data book for civil engineers, V. 2

Elwyn E. Seelye, John Wiley and Sons, Inc., New York, 3rd Edition, 1957, 600 pp., $20

One of three reference books for civil engineers, specially designed for use by chief or administrative engineers, providing data for preparation of specifications on airports, roads, railroads, bridges, dams, docks, drainage, and sewers. Includes relative cost analyses for different types of work and a classified glossary of terms. Specifications on concrete include a "brief" concrete specification, a "full" concrete specification, and specification for concrete pavements and precast curbing.

The concrete specifications include many items at variance with ACI recommendations, such as: (1) use of job-cured specimens for acceptance of concrete quality; (2) use of job-cured specimens to determine necessity for load tests; (3) only Type II cement for general use and only billet steel bars, intermediate grade, for reinforcement; (4) use of "dust-on" floor finishes; (5) prohibition of calcium chloride in structural concrete; (6) inconsistent limits on water-cement ratios for heavy-duty floor finishes; (7) a requirement that "imperfect" exterior surfaces of concrete may not be patched but must be repaired by removal and replacement of the entire member; (8) a load test using a lesser combination of test loads but somewhat the same criteria as the ACI Building Code.

Effects of nuclear weapons

United States Atomic Energy Commission, U. S. Government Printing Office, Washington, D. C., June 1957, 579 pp., $2

A complete discussion of background information necessary to the design of structures to resist the effects of nuclear weapons. It includes description of explosions; air blast phenomena and effects; structural damage; the mechanism and illustrations for surface and subsurface bursts upon surface, partially underground, and underground structures; the effects of radiation; and protective measures to secure blast resistant structures and to provide shelters for personnel against radiation and fallout.

Transformer noise—performance of sound barriers and other acoustic treatments

A. T. Edwards, *Ontario Hydro Research News* (Toronto), V. 9, No. 1, Jan.-Mar. 1957, pp. 1-8, 19

Discusses the reduction of sound in transformer installations to levels acceptable for various types of neighborhood. Various surface acoustic treatments and structural devices are described. Of particular interest is a treatment of exposed concrete masonry to increase its acoustic properties. The treatment consists of drilling a properly sized hole through the face shell of each of the cores within the blocks to form a series of Helmholtz resonators.

Manual of recommended practice for inspection and testing of concrete materials and concrete

Concrete Industry Board, Inc., New York, 1957, 12 pp., $0.50

Presents recommendations of the Concrete Industry Board of New York City for the procedures and specification of concrete testing and the employment of testing laboratories, recommended minimum test procedures; concludes with a discussion clarifying the definitions of "controlled" concrete and "average" concrete under the building laws of the City of New York. A noteworthy feature of the discussion on controlled concrete is the limitations imposed upon design strengths; maximum design strength for ordinary concrete being 2200 psi; for ordinary controlled concrete, 3500 psi; and a special maximum for controlled concrete to be used in columns when a licensed professional engineer or registered architect is stationed both at the mixing plant and at the structure, of 4000 psi.

Automatic equipment and comparative test results of the four ASTM freezing-and-thawing methods for concrete

H. T. Arni, B. E. Foster, and R. A. Clevenger, *Proceedings*, ASTM, V. 56, 1956, pp. 1229-1256

A description of automatic apparatus to operate simultaneously all four of the ASTM tentative methods of tests for resistance of concrete specimens to freezing and thawing. Data obtained from tests of six concretes made from three different aggregates and with two air contents are given. The purpose is to indicate which methods best separate the different concretes as well as how many cycles of tests were required to give a reliable separation between these concretes.

Estimating general construction costs

Louis Dallavia, F. W. Dodge Corp., New York, 2nd Edition, 1957, 197 pp., $8.50

An estimating handbook adaptable to changing unit costs and different local conditions. Provides a method of estimating in which local material costs and labor rates may be substituted, providing the total direct cost by simple arithmetic. Shift cost, output per shift, and unit cost are determined by means of tables, taking into account all variables for estimating costs on earthwork, reinforced concrete work, structural steel work, masonry and carpentry. The handbook is highly technical, prepared by a practicing estimator and contractor for use by professional estimators and novices as well. Estimating form sheets and check lists for each type of construction have been included, along with tables for masonry and concrete work which schedule unit costs in both sq-ft and cu-ft terms.

Applied strength of materials

Alfred Jensen, McGraw Hill Book Co., New York, 1957, 343 pp., $5.75

Book is planned as an elementary text for a one-semester course to introduce the subject to engineering students. The principles of strength of materials are developed, then applied to design formulas in the simplest possible manner making full use of very clear diagrams without the use of calculus. Emphasis is placed upon application to common engineering problems which might be encountered by either mechanical or civil engineers.

The book is particularly appropriate as a text for undergraduate courses.

Apparatus for an approximate hardness test of concrete

Civil Engineering and Public Works Review (London), V. 52, No. 607, Jan. 1957, p. 49
Highway Research Abstracts June 1957

There are four common methods which are used in Russia for the approximate testing of the hardness of concrete: by a pistol shot; with the blow of a standard hammer; by use of a hydraulic jack; and use of a steel ball and hammer.

In *Beton i Zhelezobeton* (No. 8, 1956) a new method is described which was developed by the Ukranian Academy of Architecture and Building. In this method a pendulum is used with a small steel ball fixed at the end. For the test, the pendulum is brought to a horizontal position and is fixed by a hook. By pressing a finger trigger, the pendulum is released and the ball strikes a steel pin. On the rebound the ball moves a pointer which indicates the position at which the rebound of the ball ceases. Tests made with 2500 specimens and compared with crushing tests showed variation from +21 percent to −15 percent.

ACI News Letter

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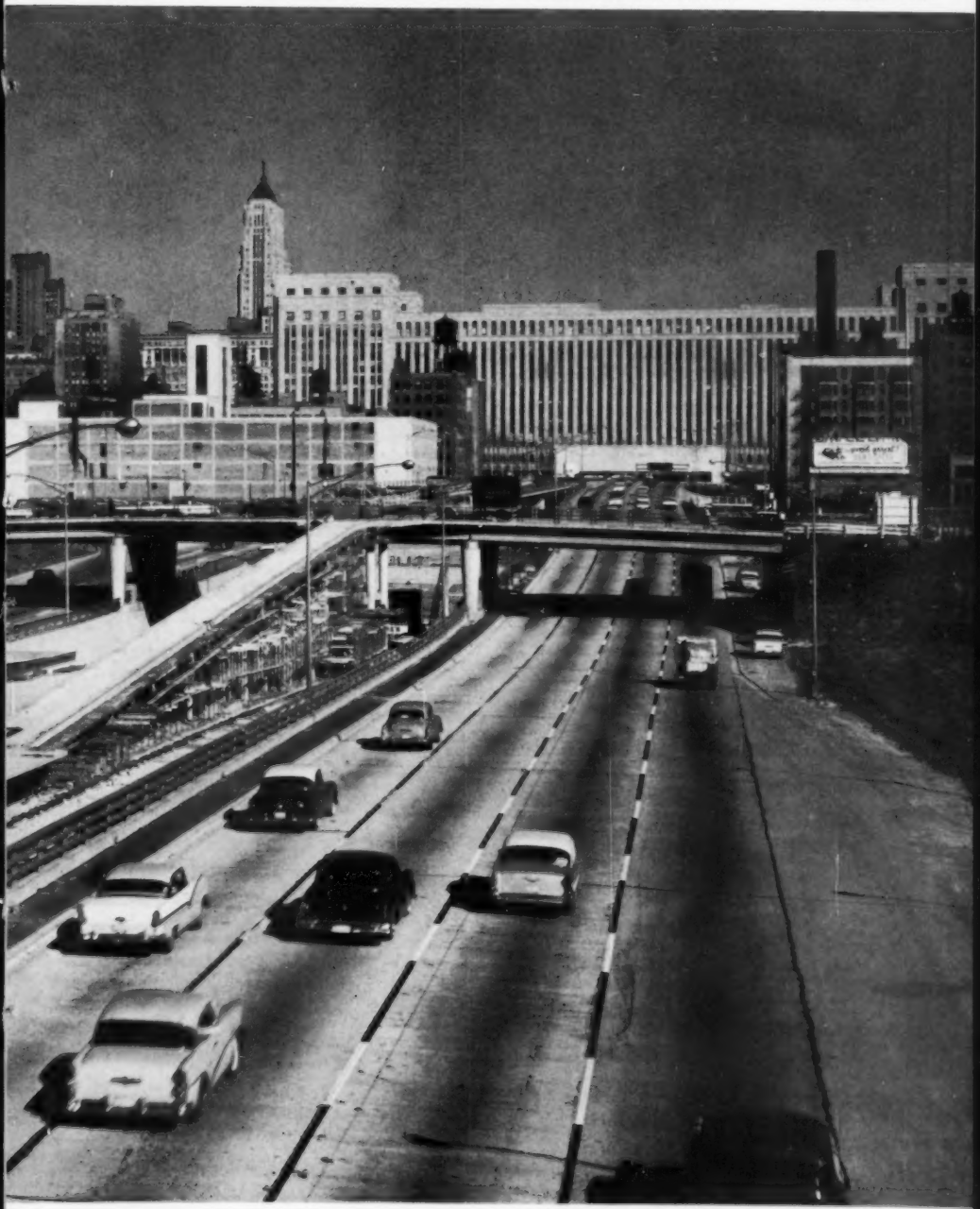

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News Letter Contents

December 1957

On the Cover—Congress Street Expressway in Chicago extending directly through the United States post office building in the heart of the city. The Chicago Board of Trade Building, one of the city's tallest, looms in the background. One of the most concentrated retail shopping sections in the world is located in the "Loop" beyond. Chicago, "Host City of the Nation," will be the site of the 54th annual ACI convention, Feb. 24-27, 1958, at the Morrison Hotel.

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ACI Awards Announced

Recognition for Gilkey, Amirikian, Copeland, Ferguson, Minges, Wild

Recipients of the 1958 ACI awards have been announced by the Board of Direction following approval of the Awards Committee report. The Turner Award goes to Herbert J. Gilkey, Iowa State College, Ames; Lindau Award to Arsham Amirikian, Bureau of Yards and Docks, Washington, D. C.; Wason Medal for Research to Phil M. Ferguson, University of Texas, Austin; Wason Medal for Most Meritorious Paper to R. E. Copeland, National Concrete Masonry Association, Chicago; and Construction Practice Award to James S. Minges and Donald S. Wild, both of Marchant and Minges, Hartford, Conn.

Awards will be presented at the 54th Annual ACI Convention to be held February 24-27, 1958, at the Morrison Hotel in Chicago.

Henry C. Turner Award to Herbert J. Gilkey

For notable leadership in advancing the knowledge of properties of plain and reinforced concrete.

This award, established in 1927 by the late Mr. Turner, ACI past president, is presented for notable achievement in, or service to, the concrete industry. The medal is of gold and is accompanied by an engraved certificate that recounts the medal's purpose.

Eminent in the field of research in engineering materials, especially concrete, Professor Gilkey has written several textbooks and manuals, as well as some 150 papers and discussions, dealing with concrete engineering, materials, and testing methods, many of which have appeared in the ACI JOURNAL. In 1939 he was awarded the Wason Medal for "the most meritorious paper" along with S. J. Chamberlin and R. W. Beal for "The Bond Between Concrete and Steel."

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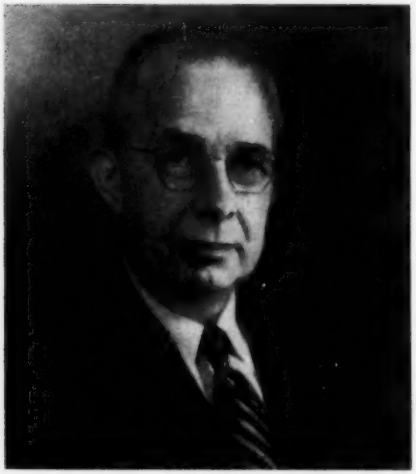


H. J. Gilkey

Culminating some 25 years of continuous activity on both technical and administrative committees, Professor Gilkey was elected president of the Institute in 1949. He had joined ACI in 1924 and in addition to serving on various technical committees in the past years, he has been chairman of the Standards Committee, Committee 208, Bond Stress, and Committee 115, Research. He was a member of the Publications Committee 1939-46; Board of Direction, 1937-48 and 1945-46; and vice-president, 1947-48.

Educated at Oregon State College, Massachusetts Institute of Technology, Harvard University, and the University of Illinois, he began his teaching and research at the University of Illinois in 1921. Continuing his work at the University of Colorado from 1923 to 1931, he then became head of the theoretical and applied mechanics department at Iowa State College, from which post he recently retired. Last year Professor Gilkey was elected to honorary membership in the American Society for Testing Materials.

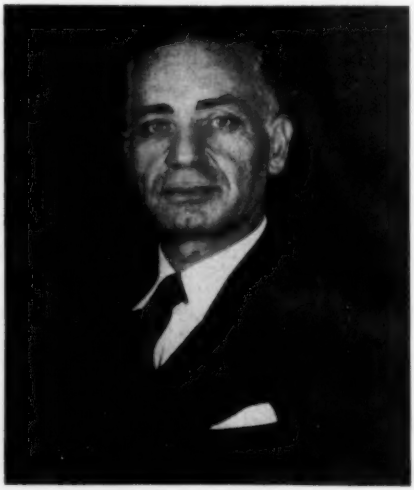
Alfred E. Lindau Award to Arsham Amirikian

In recognition of his continued devotion to the improvement of design practice for precast reinforced concrete construction.

This award was founded in 1947 by the Concrete Reinforcing Steel Institute to honor the memory of Alfred E. Lindau, past president of the American Concrete Institute, who died at Pearl Harbor, Dec. 14, 1944, while serving as chief civilian engineering assistant to the Officer in Charge, Pacific Naval Air Bases. The token of the award is a bronze plaque bearing a bas-relief portrait of Mr. Lindau. It is given only for outstanding contributions to reinforced concrete design practice and is not restricted to ACI members.

Mr. Amirikian is chief design engineer, Bureau of Yards and Docks, Department of the Navy, Washington; a great part of his work in recent years has been in the field of thin shell precast concrete construction and design of protective structures against conventional bombs and atomic blasts. He originated and developed the modern precast concrete technique utilizing thin shell cells, ribbed panels, and hollow section rigid framing. He has developed new methods of analysis and applied them to the design of structures, including such important projects as airplane hangars and industrial buildings, as well as auxiliary floating craft for the Navy, huge floating drydocks and gate caissons built of reinforced concrete during World War II.

Mr. Amirikian has been widely honored in recent years for his outstanding contributions to the development of reinforced concrete. He was awarded the First Grand Award of



Arsham Amirikian

James F. Lincoln Arc Welding Foundation in 1942; the Fuertes Graduate Medal of Cornell University in 1943, the Lincoln Gold Medal of the American Welding Society in 1949, and the Concrete Reinforcing Steel Institute Award in 1952.

Since joining the Institute in 1945, Mr. Amirikian has been a frequent contributor of technical papers for publication in the JOURNAL and has been active in ACI committee work. At the present time he is chairman of ACI Committee 324, Precast Reinforced Concrete, Thin Sections; a member of ACI Committee 116, Nomenclature, and Committee 314, Rigid Frames for Buildings and Bridges, as well as a member of the Joint ACI-ASCE Committee 323, Prestressed Reinforced Concrete; he is a nominee of the Board of Direction for a 3-year term beginning in 1958.

Wason Medal for Research

This award goes to Phil M. Ferguson for his paper "Some Implications of Recent Diagonal Tension Tests," in the August 1956 ACI JOURNAL. Founded in 1917 by the late Leonard C. Wason, this award is bestowed to an ACI member or members for the most noteworthy research or discovery reported in an Institute paper within a given publication year. This award was previously presented to Professor Ferguson, with J. Neils Thompson, for work reported in their 1953 paper "Diagonal Tension in T-Beams Without Stirrups."

Professor Ferguson is chairman of the civil engineering department at the University of Texas, where he has been a staff member since 1928. After graduating from the University of Texas in 1922, he received master's degrees from the University of Texas in 1923, and the University of Wisconsin in 1924. He spent several years with Dwight P. Robinson and Co., New York, in structural design work before returning to Texas.

Active in the Institute since joining in 1930, he is now vice-president and a member of the following technical ACI committees: 115, Research; 208, Bond Stress; 318, Standard Building Code; and ACI representative on the American Welding Society subcommittee on welding of bars, metal inserts and connections in reinforced concrete construction. Professor Ferguson is also a member of ACI's Technical Activities Committee, and heads the ASCE Committee on Masonry and Reinforced Concrete.

Wason Medal for Most Meritorious Paper

R. E. Copeland receives this medal for his paper "Shrinkage and Temperature Stresses in Masonry," in the February 1957 JOURNAL. Also established by Mr. Wason in 1917, it is presented for the most meritorious paper by a member or members of the Institute published in the latest volume of ACI *Proceedings*. This is the second time that Mr. Copeland has received the award. In 1941 he and C. C. Carlson were accorded this honor for their joint paper "Tests of the Resistance to Rain Penetration of Walls Built of Masonry and Concrete."

Mr. Copeland, director of engineering for the National Concrete Masonry Association, has been a frequent contributor of technical papers for the JOURNAL, some of which were published prior to his joining the Institute in 1939. He served in 1948 as a member of the Board of Direction and worked on the Technical Activities Committee during 1947-48 and 1953-55. He currently is a member of Committee 311, Structures of Concrete Masonry Units, and the Standards Committee on which he has served since 1953.

He attended Iowa State College and the University of Wisconsin, receiving a BS degree from the latter. Mr. Copeland served the Portland Cement Association for many years as a technical engineer before taking his present position.

Construction Practice Award

James S. Minges and Donald S. Wild have been cited for their paper "Six Stories of Prestressed Slabs Erected by Lift-Slab Methods," ACI JOURNAL, February, 1957. This construction award was founded in 1944 to help enrich the literature of that field of work and to honor the construction man—the man on the job—for his resourcefulness in translating design into a completed structure.

Mr. Minges, who joined ACI in 1956, was graduated from Rensselaer Polytechnic Insti-

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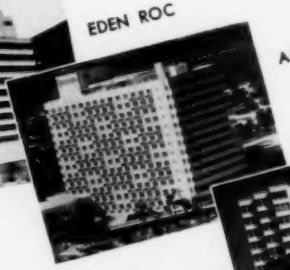
EDEN ROC



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structural engineers	Oboler & Clarke Miami Beach, Florida
general contractor	Taylor Construction Co. Miami, Florida

Time and again, reinforced concrete is the choice of leading architects for America's finest hotels and resort apartments. John Hans Graham, A.I.A., architect for the luxurious Palm Beach Towers, states, "The speed of construction, the resultant economy, and the ease of cantilevering were invaluable assets that more than justified the choice of reinforced concrete."

The Fountainbleau, Eden Roc, Americana, and the magnificent, new Palm Beach Towers were all completed on schedule. With reinforced concrete construction, costly delays and lost hotel revenue were thus avoided.

Before you build, investigate this economical, firesafe, and flexible method of construction.

CONCRETE REINFORCING STEEL INSTITUTE 38 S. DEARBORN STREET CHICAGO 3, ILLINOIS

54th Annual Convention

Complete program announced for February 24-27

An impressive program for the 54th annual meeting of ACI has been announced by the Technical Activities Committee and the general chairman of the Chicago convention committee, A. Allan Bates, vice-president for research and development, Portland Cement Association.

Convention-goers, in session at Chicago's Morrison Hotel from February 24 to 27, will hear reports on concreting operations on the St. Lawrence Seaway in a series of papers devoted to that topic; research studies on the fatigue of concrete will comprise another session, and other featured topics include construction problems, structural design—a joint session with the American Society of Civil Engineers, and highway pavement design. The annual research session will bring latest reports, many "off-the-record," of tests on concrete in progress at institutions throughout the nation.

Institute members will act on revisions in the ACI Bylaws (reported in detail on p. 32 of this month's News Letter), and will consider the adoption of new Institute standards on pavement design and bond test for reinforcement, as well proposed revisions for two existing standards, "Specifications for Concrete Pavements and Bases (ACI 617-51)" and "Minimum Standard Requirements for Precast Floor Units (711-53)." Architect Minoru Yamasaki will report on the new ACI headquarters building now nearing completion in Detroit. Following his talk, members will see a color film showing casting and erection of the building's unique folded plate roof.

Awards to be given, dinner scheduled

The Institute's Wason Medals, Turner, and Lindau Awards, and the ACI Construction Practice Award will be presented at the luncheon February 26. Details of the awards are given on p. 3 of this month's News Letter. A dinner planned by the local committee is slated for February 26, and a ladies' program concurrent with the general convention activities has been planned by Mrs. A. Allan Bates. The diverse educational and entertainment opportunities afforded by the Windy City will enable visitors to fill any gaps between technical sessions, according to W. W. Wallace,

Chicago Convention Committee

Assisting general chairman A. Allan Bates on the local committee for the 1958 convention are: H. C. Delzell, Concrete Reinforcing Steel Institute, in charge of entertainment plans; Earl Beard, Department of Public Works, City of Chicago, inspection tours; W. W. Wallace, Portland Cement Association, publicity; and George F. Long, Inland Steel Co., treasurer. H. F. Peckworth, American Concrete Pipe Association, will handle liason between ACI and the American Society of Civil Engineers, which is meeting concurrently in Chicago.

Other Chicago committee members include: William Armstrong, Material Service Corp.; C. J. Cain, Chicago Fly Ash Co.; and A. P. Christensen, Sumner Sollitt Co. R. H. Clore, Universal Atlas Cement Co., is also a committee member, as are E. M. Fucik, Harza Engineering Co.; O. W. Irwin, Rail Steel Bar Association; Jack R. Janney, American Concrete Corp.; Douglas McHenry, Portland Cement Association; and C. E. Wuerpel, Marquette Cement Mfg. Co. Mrs. A. Allen Bates heads the group planning ladies' entertainment.

Portland Cement Association, district engineer, in charge of publicity for the meeting.

The Institute's Board of Direction and Technical Activities Committee will begin their work ahead of the general sessions, with meetings Saturday and Sunday, February 22 and 23. On Monday, the 24th, numerous Institute technical committees will hold meetings throughout the day and during the evening hours. Tuesday morning the convention program participants will breakfast together before the first general session of the 1958 convention. The detailed program is given on the following pages.

TUESDAY MORNING, FEBRUARY 25

First General Session—W. H. Price, Chairman

Opening Remarks—President Price

Proposed Revision of Institute Bylaws

Proposed Standards:

Committee 208—Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars

Committee 325—Proposed Recommended Practice for Design of Concrete Pavements

Committee 617—Proposed Revision of "Specifications for Concrete Pavements and Bases (ACI 617-51)"

Committee 711—Proposed Revision of "Minimum Standard Requirements for Precast Floor Units (ACI 711-53)"

The New ACI Headquarters Building

Minoru Yamasaki, Yamasaki, Leinweber and Associates, Birmingham, Mich.

Motion picture of ACI building in process of construction

TUESDAY AFTERNOON, FEBRUARY 25

St. Lawrence Seaway—I. L. Tyler, Session Chairman

Sponsored by ACI Committee 207

Features of the St. Lawrence International Power Project

J. Neil Mustard, Hydro-Electric Power Commission of Ontario, Cornwall

Concrete Technology of the St. Lawrence International Power Project

Harry H. McLean, New York Department of Public Works, Albany, formerly concrete control engineer with Uhl, Hall and Rich, Massena, N. Y.

Features of the St. Lawrence International Seaway Project

Gordon Kidd, St. Lawrence Seaway Authority, Montreal

Concrete Technology in the St. Lawrence International Seaway Project

M. R. Smith, Corps of Engineers, Buffalo District

A tour of the Portland Cement Association's research laboratories at Skokie, a few miles northwest of Chicago's loop, will be conducted for ACI members and guests on February 27, the closing day of the 54th annual convention. These laboratories are the largest and most completely equipped in the world devoted exclusively to research on cement and concrete. First hand observations may be made of recent developments in concrete construction at PCA's fire research building now nearing completion.

WEDNESDAY MORNING, FEBRUARY 26

Fatigue of Concrete—Gene M. Nordby, Session Chairman

Sponsored by ACI Committee 215

History of Fatigue of Concrete
Gene M. Nordby, National Science Foundation, Washington, D. C.

Effect of Range of Stress and Fatigue Strength of Plain Concrete Beams
J. M. Murdock and Clyde Kesler, University of Illinois, Urbana

Fatigue Properties of Concrete Beams
Thomas E. Stelson, Carnegie Institute of Technology, Pittsburgh, and John N. Cernica, Youngstown University, Youngstown, Ohio

Probability of Fatigue Failure of Plain Concrete
John T. McCall, Michigan State University, East Lansing

Static and Fatigue Strength of Beams with Tensile Reinforcement
Tien S. Chang and Clyde Kesler, University of Illinois, Urbana

Construction—Douglas McHenry, Session Chairman

Failures of Concrete Structures
Jacob Feld, Consulting Engineer, New York

Control of Concrete Mixes
E. A. Abdun-Nur, consulting engineer, Denver, and J. J. Waddell, Joseph K. Knoerle and Associates, Chicago

Construction Features of the British Columbia Electric Building, Vancouver
O. Safir, consulting engineer, Vancouver

Wide Span Hyperbolic Paraboloidical Shell
Anton Tedesko, Roberts and Shaefer Co., New York

Continued on p. 10

Convention continued . . .

WEDNESDAY NOON—Awards Luncheon

WEDNESDAY AFTERNOON, FEBRUARY 26

Structural Design—Phil M. Ferguson, Session Chairman

A joint session with the ASCE Structural Division

Influence of Design and Details on Concrete Deterioration

P. D. Miesenhelder, Association of American Railroads, Chicago

Effect of Axial Force and Shear Strength of Reinforced Concrete

James W. Baldwin, Jr., University of Illinois, Urbana, and Ivan M. Viest, AASHO Road Test, Ottawa, Ill.

LADIES!

For those of you who plan to "do the town" while husbands advance the cause of concrete, Ruth Bates has been busy planning a schedule which includes live TV shows, teas, a style show at Marshall Field's, and a puppet opera at the famous Kungsholm Restaurant.
The program will allow ample time for window shopping (or shopping of the more serious kind) in the fabulous stores of Chicago's loop and the smart shops on Michigan Avenue.

See you in the Windy City

Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members without Shear Reinforcement

Charles S. Whitney, Ammann and Whitney, New York and Milwaukee

Development of Precast Concrete Girders Reinforced with High Strength Deformed Bars

J. R. Caston and Eivind Hognestad, Portland Cement Association, Chicago

Design of Highway Pavements—E. A. Finney, Session Chairman

Sponsored by ACI Committee 325

New Factors in Pavement Design

E. A. Finney, Michigan State Highway Commission, East Lansing

Subcommittee reports as follows:

Prestressed Concrete Pavements
Continuously Reinforced Concrete Pavements
Concrete Overlay for Concrete Pavements

WEDNESDAY EVENING—Convention Banquet

THURSDAY MORNING, FEBRUARY 27

Annual Research Session—George W. Washa, Session Chairman

Sponsored by ACI Committee 115

THURSDAY AFTERNOON, FEBRUARY 27

Tour of Portland Cement Association Laboratories

Southern California chapter of ACI conducts first meeting

The first general meeting of the Southern California Chapter of ACI, held Nov. 8, 1957, at the Rodger Young auditorium in Los Angeles, was attended by 130 members and friends.

Following an evening dinner, the general program opened with Henry M. Layne, chairman of the chapter, explaining how committee efforts, beginning in October, 1956, had resulted in the formation of the pro tempore chapter whose general membership was then meeting for the first time. Present plans do not call for regularly scheduled monthly meetings but visualize the solution of local technical problems through prompt investigation and reporting by selected teams of reporters. The first task is the revision and updating of a bulletin on tilt-up construction developed several years ago by the Structural Engineers' Association of Southern California. Complete cooperation and free exchange of information were cited as necessary to success of the chapter effort.

Institute President Price reviewed action by the ACI Board of Direction culminating on November 4 in Board approval of a petition to revise the Institute Bylaws to provide for and regulate the operation of local chapters. He emphasized that convention action and a letter ballot of the Institute membership were necessary steps to final adoption of these changes.

Past-president Charles S. Whitney, Ammann and Whitney, New York and Milwaukee, was the technical speaker for the evening. He described and illustrated the application of folded-plate cantilevered slabs in aircraft hangars where obstructions are unacceptable. Of particular interest were load tests of the cable-supported slabs.

ACI national officers attending this first meeting included: president, Walter H. Price, U. S. Bureau of Reclamation, Denver; vice-president, Douglas McHenry, Portland Cement Association, Chicago; vice-president, Phil M. Ferguson, University of Texas, Austin; directors, C. D. Wailes, Jr., Wailes Precast Concrete Corp., Sun Valley, Calif.; J. W. Kelly, University of California, Berkeley; and William A. Maples, secretary-treasurer.

Samuel Hobbs is secretary-treasurer of the southern California Chapter with headquarters in Room 1058, 742 South Hill St., Los Angeles.

Koontz named department head

Carl H. Koontz, member of the faculty of Worcester Polytechnic Institute, Worcester, Mass., since 1952, has been named head of the civil engineering department there.

Silveston named firm partner

Tippets-Abbett-McCarthy-Stratton, New York, consulting engineers, announce the admission of Barnett Silveston as general partner, and Eugene E. Halmos and Wilson V. Binger as associate partners.

Mr. Silveston has been with the firm since 1943 as associate partner in charge of port and harbor engineering. Mr. Halmos joined the firm in 1948 and has since served as chief design consultant. Mr. Binger, manager of the organization's operations in Venezuela, Colombia, Ecuador, and Central America, for the past 3 years, has been with the firm since 1952.

ACI Journal gift order form

Help your friends to keep abreast of latest developments
Give a gift subscription this year of the ACI JOURNAL

The JOURNAL is recognized as the foremost source of technical information in the United States and throughout the world. For 54 years, ACI has brought the latest developments and techniques to the readers of the JOURNAL. Institute standards and building codes represent accepted methods in concrete construction. Each month, eminent authorities in their field submit the results of their research and production methods to the ACI JOURNAL for publication. It is this exchange of ideas and facts through the pages of the JOURNAL that maintains the modern up-to-date information that provides the newest concepts in concrete construction. Every engineer, architect, manufacturer, production supervisor, contractor, and construction supervisor wants to keep abreast of the latest methods of design and construction in concrete. This information is available to him only through the pages of the ACI JOURNAL. Right now is the time to order your gift subscription of the ACI JOURNAL. Fill in the coupon; we'll do the rest.

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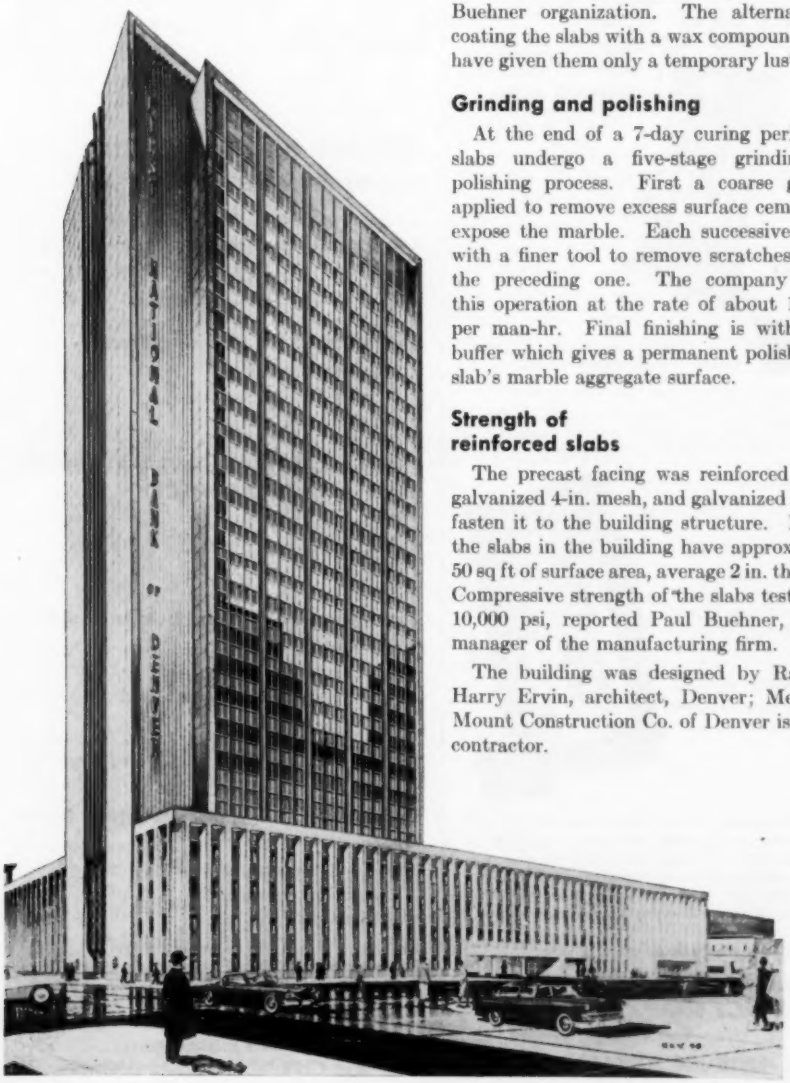
Precast Marble Concrete Facing

Over 100,000 sq ft of white marble-concrete slabs have been precast and finished by Otto Buehner & Co. of Salt Lake City, to face the new 28-story First National Bank Building of Denver now nearing completion.



The Mo-Sai precast facing used on this job was made of white Georgia marble aggregate and white cement, polished to a natural high gloss finish. The selection of an aggregate which would shine was one of the most important steps in production, according to O. C. Wilde, production engineer for the Buehner organization. The alternative of coating the slabs with a wax compound would have given them only a temporary luster.

Grinding and polishing

At the end of a 7-day curing period, the slabs undergo a five-stage grinding and polishing process. First a coarse grind is applied to remove excess surface cement and expose the marble. Each successive step is with a finer tool to remove scratches left by the preceding one. The company figures this operation at the rate of about 10 sq ft per man-hr. Final finishing is with a felt buffer which gives a permanent polish to the slab's marble aggregate surface.

Strength of reinforced slabs

The precast facing was reinforced with a galvanized 4-in. mesh, and galvanized anchors fasten it to the building structure. Most of the slabs in the building have approximately 50 sq ft of surface area, average 2 in. thickness. Compressive strength of the slabs tested over 10,000 psi, reported Paul Buehner, general manager of the manufacturing firm.

The building was designed by Raymond Harry Ervin, architect, Denver; Mead and Mount Construction Co. of Denver is general contractor.

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A steel constructed and integrally welded Concrete Tester that is calibrated for accuracy using U. S. Bureau of Standards certified calibration apparatus. The CT-900 meets ASTM and AASHO specifications for hydraulic testing machines and is accurate to within 1% of the indicated load.

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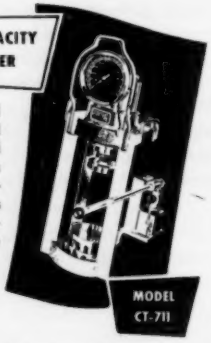

A sturdy, compact and entirely self-contained unit. Easily operated either by hand or by an electrical pump attachment. The CT-711 is also calibrated for accuracy to within 1% of the indicated load.

Many other units of Engineering Test Apparatus for Concrete Testing are available. Our New Catalog covers completely all testing equipment and accessories used in this field. Illustrated Bulletins describe in detail the Apparatus shown above.

SOILTEST Incorporated

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Awards

Continued from p. 5

tute with a BS in CE in June, 1941. He is a member of the firm of Marchant and Minges, consulting engineers, in West Hartford, Conn., who handle structural, mechanical, and electrical engineering projects:

Mr. Wild, who is engineer in charge of structural development for Marchant and Minges in West Hartford, received his CE degree from Rensselaer Polytechnic Institute in June, 1950. He also became an Institute member in 1956.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 116, Nomenclature

Morton Sherman
Zonolite Co.
Evanston, Ill.

Committee 323, Prestressed Reinforced Concrete (Joint ACI-ASCE)

W. R. Wilson
Atchison, Topeka & Santa Fe Railway
Chicago, Ill.

Committee 328, Limit Design (Joint ACI-ASCE)

A. L. L. Baker
Imperial College of Science and Technology
University of London
London, England

Committee 334, Concrete Shell Structures

Bruno Thurliman
Lehigh University
Bethlehem, Pa.

Committee 622, Formwork for Concrete

David E. Fleming
U. S. Bureau of Reclamation
Denver, Colo.

Positions and Projects

ASCE awards made

Awards for specially significant transaction papers were made by the American Society of Civil Engineers, Oct. 16, 1957, in conjunction with their annual convention in New York. Walter L. Dickey and Glen B. Woodruff, both members of ACI as well as ASCE, were awarded the James Laurie Prize for their joint paper "The Vibrations of Steel Stacks." Mr. Dickey is structural engineer for the Wailes Precast Concrete Corp., Sun Valley, Calif., and has done considerable work in earthquake design and investigation, harbor and waterfront development, the design of thin shell structures, and the use of vacuum concrete. Co-author Woodruff, a partner in the San Francisco consulting firm of Woodruff & Sampson, is a distinguished bridge engineer. He has participated in design of such notable structures as the Mid-Hudson River suspension bridge, the Huey P. Long Bridge in New Orleans, the Tacony-Palmyra Bridge across the Delaware, and San Francisco-Oakland Bay Bridge.

Third National Construction Industry Conference held in Chicago

Armour Research Foundation of Illinois Institute of Technology sponsored the Third National Construction Industry Conference, Dec. 11-12, 1957, at the Congress Hotel, Chicago.

The theme of the conference was "Creative Trends in Structural Design" heralding features of tomorrow's buildings. Among the many prominent speakers who addressed the group were several ACI members. Felix Candela, Cubiertas Ala S.A., Mexico City,

ACI Building progress report

ACI's headquarters building in Detroit has assumed a finished look as staff members prepare for the move which will put a concrete roof over their heads.

ALUMINUM SASH for the building's glass walls have been installed, and glazing is underway. Precast concrete panels have been set in place at the base of these window walls.

ELECTRICAL FIXTURES and MECHANICAL DETAILS are being installed. Heat and light are already available from the building's own system.

PRECAST GRILL of ornamental concrete has been positioned over basement window openings.

Convention goers at ACI's 54th annual convention in Chicago, Feb. 24-27, 1958, will hear architect Minoru Yamasaki's report on the unique structure, and will see motion pictures of concreting and erection of the building's precast folded plate roof.

Building Committee Report

as of October 31, 1957

ACI Building Committee shows the following contribution to the ACI Building Fund by categories as of Oct. 31, 1957.

Cement producers	$ 71,612.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,176.22
Contractors	19,445.00
Ready-mixed concrete and aggregate industry	12,548.87
Membership at large	8,597.99
Admixtures manufacturers	7,458.93
Concrete machinery and specialty products	3,585.00
Concrete products manufacturers	2,570.00
Special contributions	1,766.43
Eastern Canada	1,496.90
	$203,257.34
Pledges in hand	$ 7,300.00
GRAND TOTAL	$210,557.34

spoke on hyperbolic paraboloids, and Anton Tedesko, Roberts and Schaefer Co., New York, covered thin shells.

Joseph J. Shideler, Portland Cement Association, Chicago, addressed the materials session on prestressed and lightweight concrete, while Eivind Hognestad, Portland Cement Association, Chicago, spoke on ultimate strength design of structural concrete. Joseph P. Wolff, commissioner, Department of Buildings and Safety Engineering, Detroit, presided at the luncheon December 11.

McLean appointed director

Harry H. McLean, formerly concrete control engineer on the St. Lawrence Seaway project with Uhl, Hall and Rich, Massena, N. Y., is now director of the public works laboratory of the New York Department of Public Works, Albany. Mr. McLean, a member of ACI Committee 614, Measuring, Mixing, and Placing Concrete, is scheduled to speak on concrete technology of the St. Lawrence project at the forthcoming ACI convention, Feb. 25-27, 1958 in Chicago.

LOOKING AHEAD

January 6-10, 1958—Highway Research Board, Annual Meeting, Sheraton-Park Hotel, Washington, D. C.

January 16-18, 1958—National Concrete Products Association of Canada, Annual Convention, Chateau Laurier Hotel, Ottawa, Ont.

January 20-23, 1958—American Road Builders' Association, Annual Convention, Sheraton-Park Hotel, Washington, D. C.

January 26-30, 1958—Associated Equipment Distributors, 39th Annual Meeting, Conrad Hilton Hotel, Chicago.

January 27-30, 1958—Plant Maintenance and Engineering Show and Conference, Amphitheatre and Palmer House, Chicago, Ill.

January 27-31, 1958—University of Florida–Prestressed Concrete Institute, Second National Prestressed Concrete Short Course, Ellinor Village, Daytona Beach, Fla.

February 10-13, 1958—National Ready Mixed Concrete Association, 28th Annual Convention, Conrad Hilton Hotel, Chicago, Ill.

February 10-13, 1958—National Sand and Gravel Association, 42nd annual convention, Conrad Hilton Hotel, Chicago, Ill.

February 10-14, 1958—American Society for Testing Materials, Committee Week, Hotel Statler, St. Louis, Mo.

February 17-19, 1958—National Concrete Masonry Association, 38th Annual Convention, Chicago, Ill.

February 17-19, 1958—National Crushed Stone Association, 41st Annual Convention and Biennial Exposition of the Manufacturers Division, Hilton Hotel, Chicago, Ill.

February 24-26, 1958—National Concrete Products Association (Canada), Ninth Annual Convention, Seigniory Club, Montebello, Quebec.

February 24-26, 1958—National Concrete Contractors Association, 1st Annual Convention and 1958 Concrete Exposition, Chicago Coliseum, Chicago, Ill.

February 24-27, 1958—ACI 54th Annual Convention, Morrison Hotel, Chicago, Ill.

February 24-28, 1958—American Society of Civil Engineers, National Convention, Sherman Hotel, Chicago, Ill.

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MARACON promotes more complete hydration of cement particles and permits a substantial reduction in the unit water content without loss of plasticity or consistency of the mix. Low water content means . . .

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Pouring concrete containing Maracon in construction project at Naheola, Alabama.

With Maracon you can:

1. LOWER CONCRETE COSTS: —
   - A. Maintain slump and workability at low W/C ratios.
   - B. Attain higher strengths without increasing cement content of a mix.
   - C. Permit economical redesign of conventional concrete mixes.
2. IMPROVE CONCRETE QUALITY: —
   - A. Minimize shrinkage in concrete before and after hardening, due to lower water content and more complete hydration of cement.
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   - C. Achieve greater density and higher durability factors.

MARACON also reduces water requirements in concrete mixes containing pozzolanic materials.

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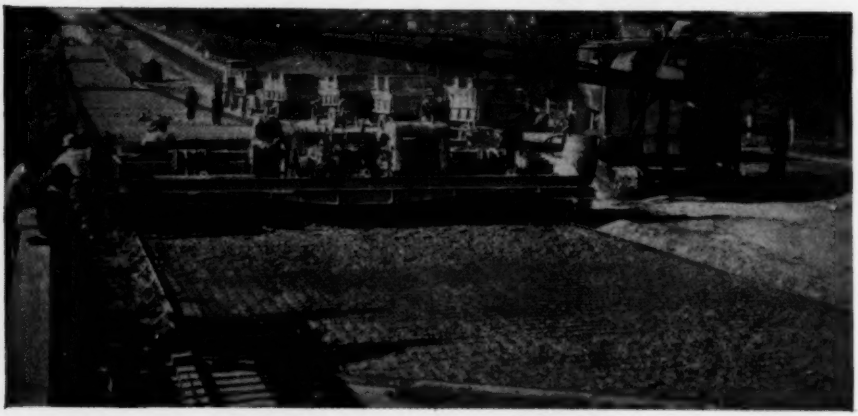

The addition of welded wire fabric to concrete slabs substantially reduces the tendency to lengthen over protracted periods of time. Research on actual highway projects indicates that steel reinforced concrete slabs increase in length only about 1/6 as much due to cracking as non-reinforced concrete slabs over a space of 10 years.*

Minimize your maintenance problems with Clinton Welded Wire Fabric. The cost is paid back many times by extended, trouble-free service life.

*Journal of the American Concrete Institute, June 1954.

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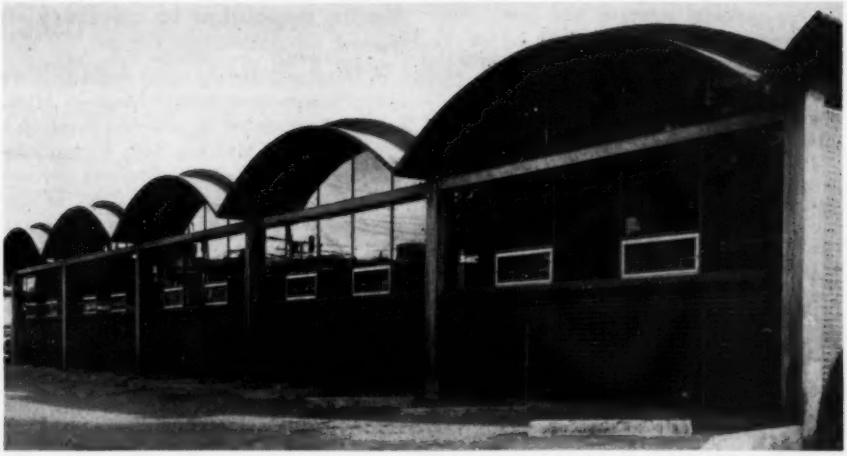

THIN SHELL CONCRETE was used as a roof for the new office building of Ketchum and Konkel, Denver consulting engineers, in a design providing excellent north light for the drafting area. The concrete barrel segments, 22½ x 48 ft, and 3¼ in. thick, are similar to some of the shells they have designed for clients.
The five shells are supported by concrete columns at the building corners and at points along the side walls where shells meet, covering an enclosed column-free area 36 x 113 ft. The end stiffeners are on top of the roof so that windows can be used in the ends of the shells. Ceiling height to the bottom of the shell is 10½ ft. East and west walls of the building are windowless, and on the south the shells overhang the wall 5 ft. The barrel segments are coated inside with acoustic plaster, outside with tar and gravel.
Ketchum and Konkel prepared the building's structural design, with an architectural scheme by Jerome K. Nagel, Denver. Construction by Hyder Construction Co., Denver.

Seelye and associates awarded contract for missile project

The Strategic Air Command of the U. S. Air Force has awarded Grad, Urbahn and Seelye, architectural-engineering joint venture, New York, a contract to prepare plans and specifications for multi-million-dollar facilities that will be required in this country's first site for launching the "Snark," intercontinental guided missile. Howard Grad, Frank Grad & Sons, Newark and Washington architectural and engineering firm, will act as managing director of the project. Besides the Grad concern, the joint venture comprises two New York firms, Urbahn, Brayton and Burrows, architects, and Seelye, Stevenson, Value and Knecht, consulting engineers.

Turner awarded hotel contract

Turner Construction Co., New York, has been selected by Hilton Hotels Corp. to build the $15 million Hilton hotel in Pittsburgh. It is anticipated that approximately 2 years will be required for the construction of the 22-story hotel situated in the Gateway Center development of Pittsburgh's Golden Triangle.

University of California schedules highway conference

The tenth annual California Street and Highway Conference is scheduled for Jan. 29-31, 1958, on the Los Angeles campus of the University of California.

Chairman for the 1958 Conference is Lewis F. Arnold, engineer of administration, City of Los Angeles. Vice-chairmen are Marshall M.

Wallace, county surveyor and road commissioner, Sonoma County, and Edwin T. Telford, assistant state highway engineer.

Cleveland conference covers concrete

The construction division of the Cleveland Engineering Society conducted its Sixth Annual Construction Conference in Cleveland last month. The program of this conference on modern industrial and commercial construction included a discussion by T. G. Landale, manager, Methods and Standards Division, M. W. Kellogg Co., New York, on "How Far Can We Go With Precast Concrete," a study of the possible economics of precasting versus casting concrete in place.

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Martin appointed to advisory board

J. Gardner Martin, district engineer, Portland Cement Association, Lansing, Mich., has been appointed to a 3-year term on a newly organized Michigan State Highway Industry Committee. This eight-man advisory committee will study such matters as construction standards, pre-qualification of contractors, and broad policy questions including route locations and types of surfaces to be used on future construction.

Mr. Martin is a charter member of the Michigan Society of Professional Engineers and the Engineering Society of Detroit. He is a member of ACI, ASCE, Citizens Advisory Panel, Legislative Highway Study Committee, and a director of the Michigan Good Roads Federation.

Giffels and Rossetti form new firm

Formation of the firm Giffels and Rossetti, Architects and Engineers, with offices in Detroit, New York, and Houston was announced recently by Raymond F. Giffels. Mr. Giffels, civil and structural engineer, and Louis Rossetti, a fellow of the American Institute of Architects, have been associated in the practice of engineering and architecture for 30 years.

Giffels & Vallet, Inc., remains in being with Mr. Giffels as president; however, new contracts in general will be undertaken in the name of Giffels and Rossetti, Architects and Engineers.

Raymond Concrete Pile promotes Kiesel

Charles B. Kiesel, Jr., has been named manager of Raymond Concrete Pile Co. domestic and foreign prestressed concrete product sales. Mr. Kiesel joined the company in 1948 as assistant general superintendent. He was a superintendent on the construction of the steam power plant and wharf project in San Juan, Puerto Rico, constructed by Raymond for the Puerto Rico Water Resources Authority. From 1953 until his new appointment Mr. Kiesel was a project manager. He will be assisted in his new post by Myers Van Buren, who has been named prestressed products engineer.

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You can economically place high quality concrete floors in cold weather by using Sikacrete Accelerating Densifier.

EARLY STRENGTH—Sikacrete is a liquid admixture which causes rapid strength development in concrete and mortar. Sikacrete saves hours of overtime finishing and reduces the possibility of damage by freezing. Floors may be opened to traffic sooner.

STRUCTURAL BENEFITS—Sikacrete contains Plastiment densifying agent which gives these structural benefits: greater density, hard non-dusting surfaces, increased ultimate strength, and reduced cracking.

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PROPORTIONS—Sikacrete proportions may be varied to suit job conditions. Normally 1½ gallons are used per cubic yard.

ECONOMY—In addition to lower finishing costs, early strength reduces both the time and cost of cold weather protection. You will increase structural quality and pocket savings greater than the cost of Sikacrete.

OTHER USES—Sikacrete is also a money-saver when laying brick or masonry units, and when placing structural concrete in cold weather. Advantages include faster job progress, improved quality, and the reduction or elimination of white deposits of efflorescence.

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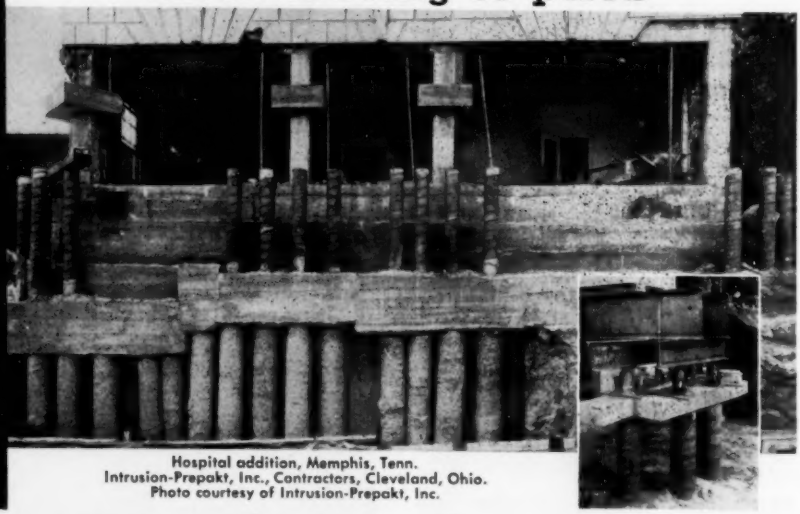
Building load temporarily transferred from footing to piles



Hospital addition, Memphis, Tenn.
Intrusion-Prepakt, Inc., Contractors, Cleveland, Ohio.
Photo courtesy of Intrusion-Prepakt, Inc.

Pile extensions formed with SONOCO SONOTUBE® Fibre Forms

Eighteen-inch diameter cast-in-place Intrusion grout piles were used as a temporary support for this hospital addition.

The pile extensions above grade were formed by Sonoco SONOTUBE Fibre Forms and took building loads off footings during underpinning with concrete slab.

Upon completion of the underpinning, the load was transferred and the pile extentions cut off.

Use low-cost Sonoco SONOTUBE® Fibre Forms for round columns of concrete and save time, labor and money. They are specifically designed to form piles, piers and underpinning for a variety of structures. Sizes from 2" I.D. to 48" I.D. up to 48' long. Can be ordered in specified lengths or sawed to requirements on the job. See our catalog in Sweets.

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Who's Who This Month

ACI Committee 711

Proposed revision of ACI 711-53, "Minimum Standard Requirements for Precast Concrete Floor and Roof Units," reported by ACI Committee 711, appears on p. 441 of this month's JOURNAL. This report has been released by the Standards Committee for publication and discussion preliminary to consideration for adoption as a revised standard by the membership at the annual convention in Chicago, Feb. 24-27, 1958.

Leo M. Legatski, University of Michigan, Ann Arbor, is chairman of the committee, which includes: Ralph W. Adams, consulting engineer, Ypsilanti, Mich.; J. H. Appleton, Alabama Cement Tile Co., Birmingham; Fred C. Bamman, F. C. Bamman Precast Concrete Corp., Hallandale, Fla., and Merwin G. Beavers, Texas Concrete Products Corp., Austin.

Other committee members are: Frank Bromilow, New Mexico College of Agriculture and Mechanic Arts, State College, N. M.; S. J. Chamberlin, Iowa State College, Ames; William C. Green, Texcrete Structural Products Co., Dallas; Harold B. Hemb, Concrete Products, Inc., Skokie, Ill.; and George E. Large, Ohio State University, Columbus.

The committee also includes F. N. Menefee, University of Michigan, Ann Arbor; C. F. Moore, Hutton and Hutton, Hammond, Ind.; O. Neil Olson, Marquette University, Milwaukee; Gayle B. Price, Price Brothers Co., Dayton, Ohio; Robert A. Rossi, Libbey-Owens Ford Glass Co., Toledo, Ohio; and R. N. Vakil, University of Wisconsin, Madison.

Jacob Feld

Jacob Feld, New York consulting engineer, presents a historical survey of concrete failures of the last half century in his paper "Failures of Concrete Structures" on p. 449 of this issue. An illustrated version of this paper is slated for the construction session of the 1958 ACI convention in Chicago, Feb. 26, 1958.

A native of Austria, Dr. Feld received his engineering education in the United States at the College of the City of New York and the University of Cincinnati. He has been active in consulting work since 1925, and has written many scientific and technical papers for German, Australian, and American periodicals. The present paper is his fourth contribution to the JOURNAL.

An ACI member since 1944, Dr. Feld has a long and impressive list of projects which he has worked on during the past 32 years. The Sixth Avenue Subway in New York, the Naval Ammunition Depot at Earle, N. J., and the Stewart Airfield at West Point, N. Y., were among his earliest assignments. One of the major projects in recent years has been a design for the New York Coliseum, along with some 50 or 60 reinforced concrete apartment houses and office buildings all of flat plate design. Among his recent, unusual concrete work was the structural design and coordination of the Guggenheim Museum now under construction in New York, for which the architectural design was done by Frank Lloyd Wright.

Discussion extended

By request, the discussion period for the October, 1957, JOURNAL paper

"Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members without Shear Reinforcement"

by Charles S. Whitney, has been extended. Discussion in triplicate should reach ACI headquarters by Mar. 1, 1958, for publication in the June 1958 JOURNAL.

Tung Au

"Ultimate Strength Design Charts for Columns Controlled by Tension," p. 479, has been submitted by Tung Au, visiting

Pouring crews can work until late afternoon without involving finishing overtime.

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with COLUMBIA CALCIUM CHLORIDE

The American Concrete Institute now recommends the addition of calcium chloride for cold weather concreting. (ACI Standard "Recommended Practice for Winter Concreting.")

Any time the temperature drops below 70°F, calcium chloride prevents lag in the set of concrete and assures you high early and ultimate strength. Forms are released faster, production schedules are accelerated, your profit on the job is increased.

Columbia Calcium Chloride may easily be added to the mix either at the plant or on the job site. It is available in two grades: High Test Flake (94-97% CaCl) in 80 lb. bags and Regular Flake (77-80% CaCl) in 25 lb. and 100 lb. bags.

Let our Columbia Calcium Chloride engineers help you save money on your fall and winter concreting. For their assistance, write today to Calcium Chloride Department at our Pittsburgh address, or at any of our fourteen District Sales Offices.

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associate professor of civil engineering, Carnegie Institute of Technology, Pittsburgh, Pa.

Professor Au received his BS from St. John's University in China in 1943, and his MS and PhD degrees, both in civil engineering, from the University of Illinois. He was a research assistant in the civil engineering department at the University of Illinois from 1948 to 1951, and subsequently was employed architectural and engineering offices, until he became assistant professor of engineering mechanics at the University of Detroit in 1955.

A registered professional engineer, and an ACI member since 1953, Professor Au is also affiliated with ASCE and ASEE.

Paul Klieger

Chapter 10 of the Long-Time Study of Cement Performance in Concrete entitled "Progress Report on Strength and Elastic Properties of Concrete," by Paul Klieger, appears on p. 481.

Mr. Klieger is senior research engineer, applied research section, Portland Cement Association, Chicago. After graduating from the University of Wisconsin in 1939, he worked for a short time with the Soil Conservation Service of the U. S. Department of Agriculture. In 1941 he joined the research staff of PCA to work on the project on which this paper is based. Army service interrupted his career from 1942 to 1945, but he then returned to the PCA research and development laboratories.

A number of technical papers prepared by Mr. Klieger have appeared in the ACI JOURNAL and the Highway Research Boards *Proceedings* and *Bulletin*. He is a member of ASTM Committee C-9 on Concrete.

René Walther

"Investigation of Multibeam Bridges," p. 505, is the contribution of René Walther, research associate, Lehigh University, Bethlehem, Pa.

The author received a degree of *Diplom Ingenieur* from the Swiss Federal Institute of Technology in Zürich in 1952 and a PhD in civil engineering from Lehigh University in 1957.

Before coming to the United States, he served 2½ years as teaching and research assistant at the Federal Institute of Testing of Materials in Zürich. Mr. Walther has been on the research staff of the Lehigh University since 1955. During this time he has been actively engaged in prestressed concrete research, studying lateral load distribution, fatigue, and shear.

Truman R. Jones, Jr., and Henson K. Stephenson

"Proportioning, Control, and Field Practice for Lightweight Concrete" appearing on p. 527 has been prepared by Truman R. Jones, Jr., and Henson K. Stephenson.

Professor Jones is associate research engineer, Texas Transportation Institute, Texas Agricultural and Mechanical College System at College Station, as well as an associate professor of civil engineering at the college. In his research capacity he is project supervisor on work related to prestressed concrete and the physical properties of structural quality lightweight aggregate concrete.

While an officer in the Corps of Engineers, 1943-45, he taught road and runway design and construction at the Engineer School, Ft. Belvoir, Va. From 1945 to 1947 he was chief of construction, Honolulu District Engineer Office, supervising $60 million dollars in

permanent construction. Leaving the Corps of Engineers, he returned to Texas A & M College as an instructor in 1947. From 1951 to 1954 he maintained a private consulting and general contracting business, but returned to the college in 1954 as associate research engineer. Professor Jones is an ACI member, member of ASCE, and serves on ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concretes.

Henson K. Stephenson, research engineer, is manager of the structural research division, Texas Transportation Institute, and also serves as professor of civil engineering at Texas Agricultural and Mechanical College, College Station. He received his bachelor's degree from Alabama Polytechnic Institute in 1923, the MS degree from the University of Michigan in 1931, and the professional CE degree from Alabama Polytechnic in 1941. His varied background of experience includes work as a structural draftsman, structural engineer, consulting engineer and senior bridge designer.

Professor Stephenson joined the staff of Texas A & M College in 1943, and assumed his present duties in 1955. His principal work in both research and teaching has been in the fields of stress analysis and structural design. An Institute member, he is active in ASCE and other professional organizations. A frequent collaborator with Professor Jones on technical papers, he is also co-author (with A. A. Jakkula) of the book *Fundamentals of Structural Analysis*.

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Two millionth ton of reinforcement

Isteg-Stahl Gesellschaft, Cologne, Germany, announced that the 2 millionth ton of *Torstahl* produced since February 1949 was rolled in June 1957. This post-war development is a reinforcement consisting of a parallel-ribbed bar, cold worked by twisting under tension to raise the yield point, resulting in a spiral-ribbed, high-yield point bar. Official German statistics show that *Torstahl* now represents 60 percent of the total annual German production, which has increased to about 950,000 tons of reinforcement.

Rolin opens consulting office

Etienne D. Rolin has opened an office in San Rafael, Calif., to serve the building industry, offering consulting services in structural engineering and soil and foundation engineering.

Bourdette R. Wood

Bourdette R. Wood, vice-president in charge of merchandising for The Master Builders Co., Cleveland, died recently.

Born in 1902, Mr. Wood was a native of Norwalk, Ohio; he graduated from Kenyon College and received his master's degree in business administration from Harvard Business School. Following employment with General Motors Corp. and an engineering firm, he joined the Master Builders staff in 1930.

An ACI member since 1949, Mr. Wood was particularly active in marketing and developing applications of Embeco, an expanding mortar product of Master Builders.

Precast and prestressed concrete units manufactured and erected by:
Edward Campbell Co., Vineland, N. J
Contractor: N. J. Post Construction Co., Oaklyn, N. J.
Architect: Philip R. Soloway, Vineland, N. J.
Consultant Designers: M. Saphier Associates, Inc., New York City

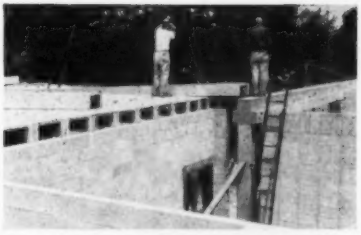
Precast concrete construction *throughout . . .*

The growing trend toward precast concrete is illustrated by this new Beth Israel Synagogue and School at Vineland, New Jersey. From standard concrete block to huge 60 ft. prestressed double tees, precast concrete serves both architecturally and structurally.

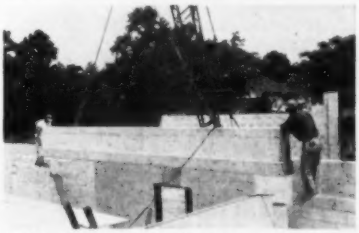
In precasting these units, the Edward Campbell Company used Lehigh Early Strength Cement for maximum production efficiency and economy. For example, in making the double tees, they used Lehigh Early Strength Cement and hot water curing. Result: early removal of units and reuse of forms in less than half the time required with regular portland cement.

This is typical of the advantages of Lehigh Early Strength Cement in modern concrete construction.

LEHIGH PORTLAND CEMENT CO.
ALLENTOWN, PA.

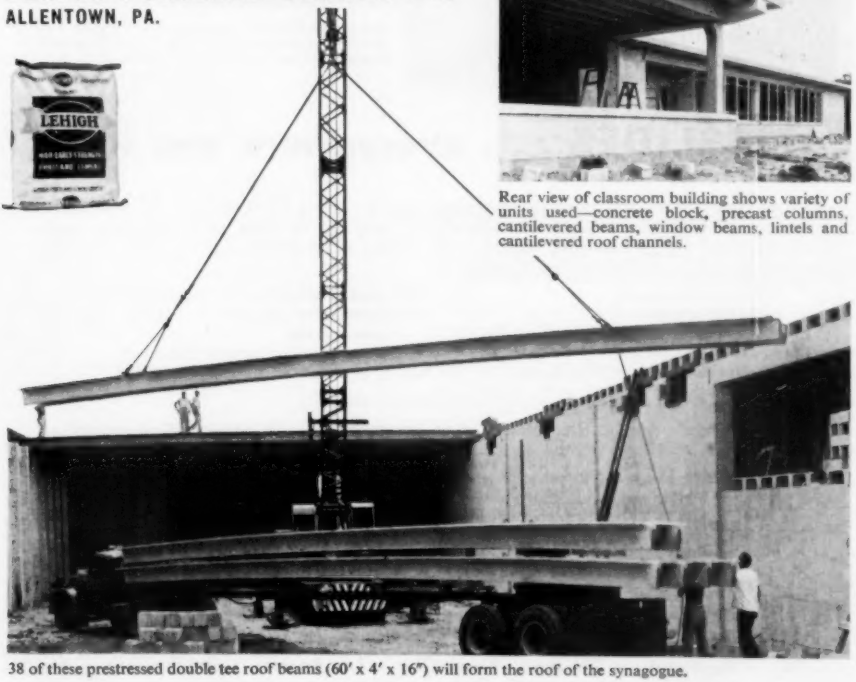


Placing one of the 440 "Clearspan" channels in the roof of the classroom building.

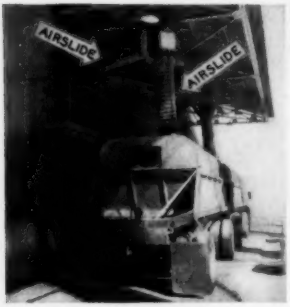


This inverted tee beam, 2'10" deep x 27' long, was especially cast for the job. Note extensive concrete masonry in common, stack and ashlar patterns. Load bearing walls include pilaster and bond beam block.

Rear view of classroom building shows variety of units used—concrete block, precast columns, cantilevered beams, window beams, lintels and cantilevered roof channels.



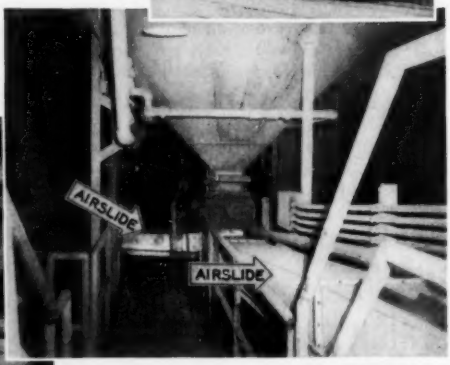
38 of these prestressed double tee roof beams (60' x 4' x 16") will form the roof of the synagogue.



Conveying cement from silos to bulk-cement truck.

Conveying cement from silos to hopper-bottom car.

Airslides conveying cement to five Fuller-Kinyon Pumps. Pumps convey cement to storage silos.



Airslide underneath storage silo. Fuller rotary valve under cone of silo for control of discharge to Airslide. Note branch Airslide, left.

AIRSLIDES . . . everywhere you look!

Everywhere you go, wherever dry fine materials are being handled, you will find more and more F–H Airslide® fluidizing conveyors being put to work. And, there are many sound reasons for its universal acceptance. What other conveyor gives you all these advantages:

- No moving parts
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- Permits flexibility of plant design
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- No large stock of repair parts
- Ease of erection, light in weight

Result: production flowing smoothly at minimum expense.

The Airslide can be used singly or in combination with other Fuller conveying systems, such as the Airveyor® or Fuller-Kinyon Pump.

If your operation involves conveying of dry fine materials, take a step forward and consult us. Fuller engineers can show you, as they have others, how the Airslide can "carry the load" in a way you never thought possible. For details write for descriptive bulletin FH-2B.

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Honor Roll

February 1—October 31, 1957

G. B. Southworth, Blas Lamberti, Arthur N. L. Chiu, and H. C. Pfannkuche are still holding the first four places on the Honor Roll.

We hope we may place your name on the Honor Roll before the close of this Honor Roll year.

G. B. Southworth	19
Blas Lamberti	15
Arthur N. L. Chiu	9
H. C. Pfannkuche	8
Luis Perez Cid	6
Phil M. Ferguson	6
Martin J. Gutzwiller	6
Walter H. Price	6
Jaime de las Casas	5
W. H. Armstrong	4
Nicandro Barboza	4
W. S. Cottingham	4
George A. Dinsmore	4
Ambrosio R. Flores	4
Louis A. Gottheil	4
Samuel Hobbs	4
Myle J. Holley, Jr.	4
J. E. Jellick	4
Franklin B. Johnson	4
Henry L. Kennedy	4
L. M. Legatski	4
Robert H. Lochow	4
James A. McCarthy	4
Frederick T. Mavis	4
Martin Schulz	4
Roberto Zepeda	4
S. E. Acker	3
James C. Amrhein	3
Luther E. Bell	3
Geo. F. Bishop	3
R. F. Blanks	3
William V. Coyle	3
C. A. Engman, Jr.	3
R. A. Kirkpatrick	3
Leo Liberthson	3
Sam C. McCluer	3
I. Martin	3
George H. Nelson	3
Wallace Sanders, Jr.	3
Harry Saxe	3
Wm. C. Schwenger	3
Howard Simpson	3
J. F. Weigel	3
H. T. Williams	3
John T. Young	3
Miles N. Clair	2½
George Brockway	2
Carlos D. Bullock	2
Jose L. Capacete	2
James Chinn	2
Dale Cobb	2
George D. Crocker	2
Roger D. de Cossio	2
C. F. DeVilbiss	2
B. M. Dornblatt	2
Andrew G. Fallat	2
E. I. Fiesenheiser	2
Kenneth H. Gedney	2
Albert Haertlein	2
Roger E. Harlepp	2
N. L. Hinkson	2
Peter S. Hobdan	2
M. W. Huggins	2
Truman R. Jones, Jr.	2
F. R. Khan	2
G. M. Kidd	2
Yavuz Kirec	2
L. Knight	2
Oscar Latorre M.	2
Daniel S. Ling	2
Harry McLean	2
Patrick McNally	2
A. J. Macchi	2
Ronald A. Maina	2
Gerald Milsom	2
Izuo Miyashita	2
Otto H. Monch	2
John M. Muir	2
D. F. Nash	2
Richard A. Parmelee	2
Jerome M. Raphael	2
Abdur Rahman S. Rasul	2
Theodore O. Reyhner	2
Sabri Sami	2
Harold Schweitzer	2
Marcel V. Sous de C.	2
Joaquin Spinel L.	2
J. P. Thompson	2
E. W. Thorson	2
Ellis S. Vieser	2
Joseph J. Waddell	2
James L. Watson	2
Robert A. Williams	2
R. A. Williamson	2
D. K. Woodin	2

New Members

The Board of Direction approved 68 Individual applications, 4 Corporations, 17 Juniors, and 19 Students making a total of 108 new members. Considering losses due to resignations, deaths, and nonpayment of dues, the total membership on Nov. 1, 1957, was 9101.

Individual

ALFORD, LYNN O., Cincinnati, Ohio (Mech. Engrg. Group Leader, Allstates Design & Development Co.)

ALVAREZ O., PEDRO FELIPE, Caracas, Venezuela (Asst. Engr., Instituto Ciudad Universitaria)

AU, EDWARD WANMAN, Singapore, Malaya (Struct. Designer, Palmer & Turner, Archs., & Engrs.)

BARTELDES, OTTO E., Chicago, Ill. (Struct. Engr., International Harvester Co.)

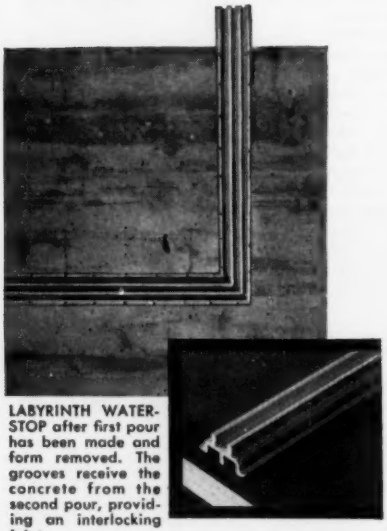
BAXTER, LESLIE W., Wichita, Kans. (Prod. Engr., PCA)

BLAKE, PAUL GIRARD, Buffalo, N. Y. (Foundation & Struct. Design, PCA)

BRETTLE, H. J., Sydney, Australia (Supv. Lecturer, School of Civil Engrg., N. S. W. Univ. of Technology)

BYRON, JAMES A., JR., Harrisburg, Pa. (Bridge Design Engr. III, Pennsylvania Dept. of Hwys., Bridge Unit)

Waterstop *in place in seconds!*



LABYRINTH WATERSTOP after first pour has been made and form removed. The grooves receive the concrete from the second pour, providing an interlocking joint.

Just a few seconds were needed to nail this LABYRINTH WATERSTOP to the form... just a few seconds and water seepage worries were over before they could ever have a chance to start.

LABYRINTH WATERSTOP forms a waterproof bond between two pours. The corrugated ribs bond firmly with the concrete.

LABYRINTH WATERSTOPS are made of flexible polyvinyl plastic... that has superior weathering qualities, is not affected by temperature changes and chemical activity.

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CHICAGO 6, ILLINOIS
Dept. 5

Made in Canada for J. E. Goodman Sales Ltd. Toronto, Ontario

CALDWELL, ERNEST O., Long Beach, Calif. (Cons. Engr.)
CARINO, ISIDRO DIMARANAN, Quezon City, Philippines (Designing Engr., Bureau of Public Works)
DEMITRACK, PAUL A., New York, N. Y. (Struct. Designer)
DOERMANN, RICHARD J., San Gabriel, Calif. (Chf. Engr. & Matls. Engr., San Gabriel Ready-Mix & Century Rock)
DORFMAN, MEYER, Flushing, L. I., N. Y. (Struct. Engr., Andrews & Clark)
DOYLE, LOUIS C., Lower Hutt, New Zealand (Engr. Mgr., Lower Hutt Plant, Certified Concrete Ltd.)
ERWIN, JAMES M., New Delhi, India (Mech. Eng., International Cooperation Mission)
FARWELL, JAMES H., San Pedro, Calif.(Vice-Pres., Super Concrete Emulsions, Ltd.)
GORDON, ALEX J., Cardiff Wales, Great Britain (Arch.)
GRAHAM, RUPERT F., Chicago, Ill. (Struct. Engr., Bureau of Engrg., City of Chicago)
HAYAMA, GEORGE K., Honolulu, Hawaii (C. E., Civil Aeronautics Admin.)
HEEREMA, PIETER SCHELTE, Maracaibo, Venezuela (C. E., Pres., C. A. Construction Heerema)
HERTEL, EUGENE A., Owensboro, Ky. (Bridge Engr., Johson, Depp & Quisenberry, Cons. Engrs.)
HOLBEKI, KAI, Maple, Ontario, (Pres., Standard Prestressed Structures Ltd.)
HONEGGAR, JOHN R., Rutherford, N. J. (U. S. Repres., Robert Aebi Co.)
ISMEN, ISMAIL, Istanbul, Turkey (Cons. Engr., Partner, Birim Muhendislik Burosu)
KALVE, ERNEST, Maywood, Ill., Struct. Designer, Naess & Murphy, Engrs. & Archs.)
KING, RALPH W., San Marino, Calif. (Struct. Engr., The Ralph M. Parsons Co.)
KNIGHT, WILLIAM D., JR., Roswell, N. M. (Assoc., Frank M. Standhardt, AIA)
LAKOS, JANOS, Rivera, Calif. (Designer, Kaynar Mfg. Co., Inc., Constr. Div.)
LEWIS, ROBERT W. H., Chicoutimi, Que., Canada (Field Engr., Arthur G. McGee & Co. of Canada)
LUEKE, GERALD L., Baltimore, Md. (Struct. Engr., Lloyd E. Mitchel, Inc.)
LYALL, JAMES R., Vancouver, B. C., Canada (Struct. Engr., City Engineers Office)
LYNCH, JOHN, Montreal, Que., Canada (Concrete Coordinator, St. Lawrence Seaway Authority)
MARMION, KEITH R., Lubbock, Texas (Asst. Prof., Dept. of Civil Engrg., Texas Technological College)
MARQUEZ P., EDUARDO J., Maracaibo, Venezuela, Struct. Engrg., Suelos Concreto-Asfalta)
MATTOCK, ALAN HANSON, Chicago, Ill. (Devel. Engr., PCA)
MEARES, HOWARD G., JR., Redding, Calif. (Proj. Engr., Shea, Kaiser, Morrison)
METTI, BISMARK A., Baghdad, Iraq (C. E., Bismark Engrg.)
MIELE, ANTHONY A., Warren, Ohio (Miele Engrg. Service)
MURPHY, G. R., San Francisco, Calif. (Struct. Design Chf., Western Distr., H. K. Ferguson Co.)
MOHD, LT. COL. SHER, Rawalpindi, Pakistan (Rtd. S. E.)
MYRDAL, ROBERT W., San Diego, Calif. (C. E., Paderewski, Mitchell & Dean)
NEDDERMAN, WENDELL H., College Station, Texas (Prof. Civil Engrg., A & M College)
NELSON, VICTOR W., Philadelphia, Pa. (Struct. Designer, R. E. Lamb, Inc.)
PALMIERI, DR. MARIO, Beverly Hills, Calif. (C. E., Mario Palmieri & Assocs.)
PANDYA, NAVIN N., Detroit, Mich. (Grad. Stud., Wayne Univ.)
PERRIN, LOUIS H., Hackensack, N. J. (Struct. Engr, Wigton-Abbott Corp.)
PERSONS, HUBERT C., Oak Park, Ill. (Tech. Writer)
RAMIREZ, SERGIO, Caracas, Venezuela (C. E., Edicar C. A.)
RESTREPO S., LUIS G., Medellin, Colombia (Struct. Engr., Ingenieria y Construcciones Ltda.)
RICHARDS, A. K., Coshocton, Ohio (Cons., Industrial, Steel & Power Plants)
RINDER, IAN J., Adelaide, South Australia (Mgr. S. A., Cement Aids)
REMMERS, JOHN, Urbana, Ill. (Research Asst., Univ. of Ill., Dept. of Theoretical & Applied Mechanics)
SAHLMAN, JOHN W., Minneapolis, Minn. (Cons. Struct. Engr.)

SARIDAN, JOHN S., Chicago, Ill. (Librarian, PCA)
SFAT, PETER, Grosse Pointe Farms, Mich. (Partner, Mortenson & Sfat, Cons. Engrs.)
SHAW, DONALD F., Los Angeles, Calif. (Proprietor, Gen. Bldg. Contr.)
SIPPEL, SEYMOUR, Brooklyn, N. Y. (Struct. Engr., Severud-Elstad-Krueger & Assocs.)
SKINNER, ROBERT LEE, Umatilla, Fla. (Sales, Master Builders Co.)
SPINDLER, MARVIN R., El Segundo, Calif. (Tech. Dir., Techkote Co., Inc.)
STILLEY, SYDNEY HIRAM, Jacksonville, Fla. (Cons. Prof. Engr.)
TANG, T. KENNETH, Seattle, Wash. (Civil & Struct. Designer, Worthington & Skilling)
THOMPSON, FRANK BLASHFORD, Montreal, Que., Canada (Account Exec., Foster Advertising Limited)
THORODDSEN, SIGURDUR S., Reykjavik, Iceland (S. Thoroddsen Engrg. Office)
TIBBITS, BURTON A., Springfield, Mo. (Struct. Engr., Harold A. Casey & Assoc.)
TING, RAPHAEL E., Detroit, Mich. (Engr., Giffels & Vallet, Inc.)
VOORHIES, LOUIS J., JR., Baton Rouge, La. (Sales Mgr., Dolese Concrete Co.)
WENTZEL, HAROLD A., Dallas, Texas (Chilean Struct. Engr., Anchor Metals, Inc.)
WHENHAM, J. E., Winnipeg, Man., Canada (Chf. Struct. Engr., Moody & Moore, Archs.)

Corporation

HOLMES & NARVER, INC., Los Angeles, Calif. (David L. Narver, Jr., Asst. Vice-Pres., Engrg. & Constr.)
FRANK J. KNIGHT COMPANY, Center Line, Mich. (James T. Curry, Gen. Mgr., Transit-Mix Concrete Div.)
MID-SOUTH ENGINEERING CO., INC., Knoxville, Tenn. (John Albert Steel, Bridge Engr.)
PINSENT CONCRETE LTD., Stephenville, Nfld., Canada (Scott Pinsent, Field Supt.)

Junior

BLOCK, MORRIS, Winnipeg, Man., Canada (Matls. Engr., Manitoba Hydro Electric Board)
BRAKS, J. RICHARD, Arlington, Mass. (Designer, Jackson & Moreland, Inc.)
CARLSON, ROLAND H., Albuquerque, N. M. (Staff Member, Sandia Corp., Sandia Base)
DE NORONHA FILHO, ANTONIO ALVES, Boston, Mass. (Struct. Designer, Jackson & Moreland, Inc.)
FALK, STEPHEN H., New London, Conn. (Constr. Supt., W. J. Barney Corp.)
GEU RIVERA, SANTOS, San Isidro, Lima, Peru (C. E., Universidad Nacional Ingeniera)
HIRONAKA, YOSHITO, Honolulu, Hawaii (C. E., Distr. Pub. Wks. Office, 14th Naval Distr.)
KAURANEN, AARNE O., New York, N. Y. (Concrete Insp., Metcalf & Eddy & LaPierre, Litchfield & Partners)
KNOX, JOHN R., New Canaan, Conn. (Engr., Praeger-Kavanagh)
KOEPSELL, PAUL L., College Station, S. D. (Asst. Prof., Civil Engrg., South Dakota State College)
KRINER, R. W., Jacksonville, Fla. (Service Engr., Lehigh Portland Cement Co.)
LABORDE, FRANCISCO JAVIER, Urbana, Ill. (Grad. Stud., Univ. of Ill.)
PACHECO, RAFAEL ENRIQUE, Caracas, Venezuela (Engr., DOF C. A. De Construcciones)
QUINTOS, ANGEL C., San Juan, Rizal, Philippines (C. E., Struct. Designer, Ambrosio Flores)
TEMPLE, R. A., Toronto, Ont., Canada (Design Engr., Wallace-Carruthers & Assocs.)
WILLIAMS, RONALD D., New York, N. Y. (Jr. Bridge Design Engr., Parsons, Brinkerhoff, Hall & Macdonald)
YU, CHARLIE, Pasay City, Philippines (Gen. Constr. Contr.)

Student

ATTAR, AHMAD M., Austin, Texas (Univ. of Texas & Reinf. Detailer Tips Iron & Steel Co. Part Time)
BERKOWITZ, MARTIN H., Forest Hills, N. Y. (Columbia Univ. & Struct. Engr. & Designer, Praeger-Kavanagh)
BRODERICK, JAMES RICHARD, Austin, Texas (Univ. of Texas)
CHACON, LUIS M., Ithaca, N. Y. (Cornell Univ.)
DAVIS, HAROLD A., Berkeley, Calif. (Univ. of Calif.)
FRADINGER, RAUL, Urbana, Ill. (Univ. of Ill.)
GERMANETTI, ENRICO, Medellin, Colombia (Facultad Nacional de Minas)
HAWKINS, NEIL MIDDLETON, Champaign, Ill. (Univ. of Ill.)
HENSON, JAMES D., Dearborn, Mich. (Wayne State Univ.)
HEYMANN, JOHN P., JR., Minneapolis, Minn. (Univ. of Minn.)
JARVIS, RICHARD L., Ithaca, N. Y. (Cornell Univ.)
JOHNSON, SVEN OLOF, Chicago, Ill. (Northwestern Univ.)
JUMBALA, CHATURISAN, Ann Arbor, Mich.
KULKA, CARLOS A., Maracaibo, Venezuela (Univ. Nacional del Zulia)
LARES R., RODOLFO, Maracaibo, Venezuela (Univ. Nacional del Zulia)
MEHTA, RAMESH J., Urbana, Ill. (Univ. of Ill.)
MERENFELD, JACOBO, Edo. Mirando, Venezuela (Univ. Nacional de Venezuela)
POPHAM, WAYNE R., Austin, Texas (Univ. of Texas)
QURESHI, SHUJAAT-ULLAN, Ithaca, N. Y. (Cornell Univ.)

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Marks joins staff of Lockwood, Andrews & Newman

C. R. Marks, III, has joined the staff of Lockwood, Andrews & Newman, consulting engineers of Victoria, Houston, and Corpus Christi, Tex.

Mr. Marks has designed dams, locks, and hydroelectric plants for the Tennessee Valley Authority and the Republic of Turkey. Since 1945 he had been chief design engineer, vice-president, and head of the Texas office of Ambursen Engineering Corp.

Errata

The following corrections should be made in "Bearing Capacity of Concrete," by William Shelson, which appeared in the November 1947 JOURNAL.

p. 410—In Column 5, Table 3, delete the ‡ following "Compressive," and insert it after the first number in the column, "3970."

p. 410—Insert § following the last number in Column 5, Table 3; add the following footnote to Table 3:

§Cylinder test piece (6 x 12 in.)

Proposed Changes In ACI Bylaws

Notice is hereby given to the membership of the American Concrete Institute that at the Institute's 54th Annual Convention, Chicago, Ill., Feb. 24-27, 1958, consideration will be given to a petition for Bylaws amendments.

The undersigned members of the American Concrete Institute hereby petition the Board of Direction to submit to the membership for adoption, in accordance with Article VI of the Bylaws, the following proposed changes in the Bylaws. (The left-hand column repeats the present wording of the Bylaws; the right-hand column the proposed rewording. Omitted unchanged sections are indicated by asterisks):

ARTICLE I—MEMBERS

PRESENT

Sec. 3. A Member shall be an individual.

A Corporation Member shall be a firm, corporation, society, agency of government or other organization.

A Contributing Member shall be an individual, firm, corporation, society, agency of government or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than an individual, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be less than 28 years of age and a registered student at a recognized technical or engineering school.

* * *

Sec. 5. All Members shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. This status of a Student Member shall change automatically to that of Junior Member on the first anniversary of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

* * *

PROPOSED

Sec. 3. A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency of government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

* * *

Sec. 5. All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversary of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

* * *

ARTICLE II—OFFICERS

Section 1. The officers shall be a President, two Vice-Presidents, twelve Directors, the three latest living Past Presidents who continue to be Members of the Institute, and the Secretary-Treasurer. The President, the Vice-Presidents and the Directors shall be elected from the Institute membership. The Secretary-Treasurer shall be appointed by the Board of Direction.

* * *

Sec. 3. Before September 1 of each year the Committee on Nominations shall report to the Secretary-Treasurer of the Institute the candidates nominated for offices to become vacant at the next annual convention and twenty candidates for membership on the Committee on Nominations which is to serve in the

Section 1. The officers shall be a President, two Vice-Presidents, twelve Directors, the three latest living Past Presidents who continue to be Members of the Institute, and the Secretary-Treasurer. The President, the Vice-Presidents, and the Directors shall be elected from the Institute membership. When a Director is elected Vice-President or President, his office as Director shall become vacant. The Secretary-Treasurer shall be appointed by the Board of Direction.

* * *

Sec. 3. Before September 1 of each year the Committee on Nominations shall report to the Secretary-Treasurer of the Institute the candidates nominated for offices to become vacant at the next annual convention and 20 candidates for membership on the Committee on Nominations which is to serve in the following year

following year. In the selection of candidates for Directors, the Committee on Nominations shall have due regard for diversity of professional and geographical representation. Each candidate for Board membership must have given consent to his nomination before the report is published. The Secretary-Treasurer shall cause notice of all such nominations to be transmitted to the membership of the Institute at least 120 days prior to the next ensuing annual convention. By petition to the Board of Direction signed by at least ten members of the Institute, within 30 days thereafter, additional nominations for offices or for membership on the Committee on Nominations may be made.

The complete list of nominations shall be submitted 60 days before the next annual convention to the Institute membership for letter ballot to be canvassed at 5 p.m. on the first day of the convention and the result announced at a session of the convention on the second day. The candidate for any office receiving the most votes shall be declared elected and the candidate receiving the most votes for membership on the Committee on Nominations shall be chairman of that committee; the four next highest shall be declared elected members of the committee. With these five the three past-president members of the Board of Direction shall serve, making a total membership of eight.

Should any member of the Committee on Nominations thus chosen fail, within fifteen days of formal notice from the Secretary-Treasurer, to make written acceptance of service, a vacancy shall occur to be filled by the candidate receiving the next greatest number of votes and so on until the five elected places on the committee shall be filled.

Sec. 4. Terms of office shall be as follows: President, one year; Secretary-Treasurer, one year; Vice-President, two years, with one Vice-President elected in alternate years; Directors, three years, with four Directors elected each year. A year is here construed as the period between the reports of tellers on canvass of ballots for Board members at two successive annual conventions.

Sec. 5. The President shall be ineligible for re-election to the same office until the lapse of at least one term. Vice-Presidents and Directors shall be ineligible for re-election to the same office until the lapse of at least one year.

* * *

In the selection of candidates for Directors, the Committee on Nominations shall have due regard for diversity of professional and geographical representation. Each candidate for Board membership must have given consent to his nomination before the report is published. The Secretary-Treasurer shall cause notice of all such nominations to be transmitted to the membership of the Institute at least 120 days before the next ensuing annual convention. By petition to the Board of Direction signed by at least ten members of the Institute, within 30 days thereafter, additional nominations for office or for membership on the Committee on Nominations may be made.

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Should any member of the Committee on Nominations thus chosen fail, within 15 days of formal notice from the Secretary-Treasurer, to make written acceptance of service, a vacancy shall occur to be filled by the candidate receiving the next greatest number of votes and so on until the five elected places on the committee shall be filled.

Sec. 4. Terms of office shall be as follows: President, one year; Vice-President, two years, with one Vice-President elected each year; Directors, three years, with four Directors elected each year. A year is here construed as the period between the reports of tellers on canvass of ballots for Board members at two successive annual conventions.

Sec. 5. A President, Vice-President, or Director, having served a full term after being elected to that office, shall be ineligible for re-election to the same office until the lapse of at least one year.

* * *

ARTICLE III—MEETINGS

Section 1. The Institute shall hold an annual convention and such other meetings as may be authorized by the Board of Direction. The time and place of all meetings shall be fixed by the Board of Direction. Notice of this action shall be sent to all members at least thirty days previous to the date of each meeting.

* * *

Section 1. The Institute shall hold an annual convention and such other meetings as may be authorized by the Board of Direction. The time and place of all meetings shall be fixed by the Board of Direction. Notice of this action shall be sent to all members at least 30 days previous to the date of each meeting. Publication in the Institute JOURNAL shall be construed to be adequate notice.

* * *

ARTICLE V (added)—LOCAL CHAPTERS

Section 1. Local chapters to provide a means of advancing the interests of the Institute in a specified geographical area and of furthering the chartered objectives for which the Institute is organized may be authorized, from time to time, by the Board of Direction upon such terms and conditions as it may deem advisable.

ARTICLE V—Continued (Note that all of this is proposed)

Sec. 2. The Board of Direction shall define the boundaries of chapter areas, within or without the confines of the United States, in which the local chapter shall be authorized to carry on its activities. Boundaries of chapter areas may be changed, from time to time, by the Board of Direction at its discretion, provided that prior notice is given to the chapter or chapters involved.

Sec. 3. Organization of a chapter may be authorized by the Board of Direction upon the written request of 25 members of the Institute residing within the proposed chapter area, provided that at least 50 members reside in the area it is proposed to encompass.

The Board of Direction may at any time terminate the existence of any chapter when in its judgment the interests of the Institute make such action desirable.

Sec. 4. Each Institute member shall be deemed eligible to belong to a chapter if his permanent address of record at Institute headquarters is within the confines of that chapter area. A member who does not maintain active membership in the local chapter, as defined by the chapter bylaws shall have no part in the government thereof. Members not resident in the geographical area of a given local chapter may become members thereof if the bylaws of the chapter permit, but shall not be entitled to vote or to hold office in such chapter.

Chapter dues shall not exceed one-half of the Institute dues for corresponding classifications of membership. Contributing Members and Corporation Members of the Institute may hold chapter memberships of the same classification, or as Members. Chapters may accept contributions to assist in financing activities which are in accord with their approved bylaws.

Sec. 5. Proposed bylaws of local chapters and amendments thereto shall be in conformity with the chartered aims and objectives and also the bylaws of the Institute and shall be approved by the Board of Direction before becoming effective.

Local chapters shall conduct only such activities as shall conform to and be in accordance with the chartered aims and objectives of the Institute.

Local chapters shall not speak, or attempt to speak for the Institute as a whole in any matter unless specifically so authorized by the Board of Direction; and further shall not incur or attempt to incur financial obligations of any kind binding upon the Institute.

Sec. 6. Each chapter shall annually submit to the Board of Direction a report of its activities and finances. Annual reports by chapters shall reach the Secretary-Treasurer of the Institute not later than January 30 of the following year.

Sec. 7. The President, or an officer of the Institute designated by him, shall make an annual visit to each chapter to confer with its officers on problems related to operation of the chapter.

Sec. 8. Funds of the Institute may be allotted to each chapter on a basis determined by the Board of Direction, the formula adopted being applicable to all chapters. Payment of funds shall be subject to regulations established from time to time by the Board of Direction.

ARTICLE VI—STANDARDS

* * *

ARTICLE VII—AMENDMENTS

* * *

Signed by:

Walter H. Price	C. D. Wailes, Jr.	Clayton L. Davis	Paul F. Rice
C. S. Whitney	C. H. Scholer	Robert C. Johnson	H. C. Delzell
Douglas McHenry	Bryant Mather	W. J. McCoy	William A. Maples
Bruce E. Foster	Phil M. Ferguson	J. W. Kelly	

The foregoing petition proposes changes and clarification in the Bylaws as follows:

1. In Article I—Members, definition of members is clarified by using the term "person" in all instances. In Section 5, wording is added to cover those cases where, because of age at graduation, a student member would advance directly to the classification of Member.

2. In Article II—Officers, a sentence is added in Section I "When a Director is elected Vice-President or President, his office as Director shall become vacant." to clear up a point on which present Bylaws are silent.

Wording in the second paragraph of Section 3 is revised to provide for canvassing of ballots on the first day of the convention without restriction as to the exact hour.

The revised Section 4 omits mention of the Secretary-Treasurer. This is to conform to intent of 1952 revision which removed him from the Board of Direction and made the term of office indefinite.

Section 5 restates eligibility for re-election to office in simpler terms.

3. In Article III—Meetings, the method of notification of meetings is more definitely stated in Section 1.

4. Article V—Local chapters, is entirely new text making possible the establishment of local chapters of the Institute.

5. Old Article V becomes Article VI and old Article VI becomes Article VII because of the insertion of new Article IV. No changes in wording of these two articles.

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Expansion joint manual

"Design Practices and Uses of Premolded Joints in Concrete Pavements," a 24-page technical booklet, contains brief general discussion of joints and their uses in concrete pavements and slabs. Fundamentals of volume change in concrete pavements and the stresses associated with it are covered.

Premolded joint materials are described in detail—including premolded joint fillers (bituminous, bituminous-fiber, and cork-asphalt types), premolded strips, premolded tongue-and-groove fillers, joint sealers, dowel bars, tie bars. Appendix covers detailed specifications for materials mentioned.

Design recommendations are given relative to longitudinal, transverse contraction, transverse expansion, and construction joints for all types of concrete pavements. This section roughly parallels the material on installation of joints in "Specifications for Concrete Pavements and Bases (ACI 617-51)," and agrees with it in some details. Abundant detail drawings and photographs of the bulletin will be of value to those unfamiliar with equipment and materials for joint installation.

Step-by-step construction procedures for installing joints are given, and a brief section treats joint maintenance.—Expansion Joint Institute, 121 Hill Ave., Aurora, Ill.

Solventless vinyl adhesive

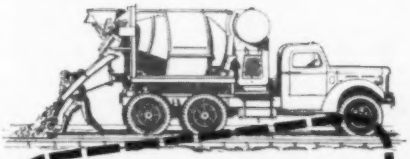
Epocast 126, a solventless vinyl adhesive, specifically developed for use with vinyl compounds, offers unique welding features, making it possible to bond rigid as well as most semi-rigid sheets of vinyl to concrete, steel, alumjnum, masonite, wood, and other surfaces, according to the manufacturer.—Furane Plastics, Inc., 4516 Brazil St., Los Angeles, Calif.

Tempering water auxiliary system

Front and rear-end tempering water system is the new design feature of the Model M series Blaw-Knox Trukmixers. The system permits the operator to introduce mix water through the charging end of the

mixer as well as through the drum head. According to the manufacturer the principal advantage in adding water through the charging hopper on high discharge

No time lost when concrete is



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mixers is that it eliminates the tendency of concrete to build up on mixer blades at the drum opening as the tempering and flush water is jetted against the blades. Auto-stop meter is furnished on all sizes of this model which is equipped with both mix and flush water tanks. The meter measures all mix water entering the drum. Another addition is an intermediate chute extension 3 ft long, which permits the concrete to be discharged at a greater distance from the mixer. —Blaw-Knox Co., 300 Sixth Ave., Pittsburgh, Pa.

Outdoor fork truck

An outdoor truck of 20,000 lb capacity with dual-drive tires has been added to the pneumatic-tire models in the Clarklift line of fork trucks. The CY-200 features "50-50" weight distribution, a planetary drive axle, and 10.00 by 20 in. tires for driving power through mud and sand and over rough terrain. The unit has continuous follow-up type hydraulic power steering which operates throughout the engine speed range to permit fast, easy turning at any speed. It will travel more than 20 mph loaded in both forward and reverse, and will climb a 22.7 percent grade loaded, according to the manufacturer.—Industrial Truck Division, Clark Equipment Co., Battle Creek, Mich.

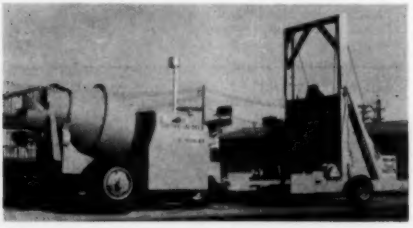
Prestressed concrete tank bulletin

Preload Bulletin T-19 on the design of prestressed concrete tanks, gives 8 pages of engineering data and formulae based on Timoshenko's *Theory of Plates and Shells*. Reinforced concrete portions of design are in accord with ACI Building Code.

The bulletin discusses ideal floor for surface tank; various types of joints used between walls and floor of prestressed tanks (including rubber pad joint); walls (short and long shell); design requirements; dome, flat, and special roof types. Details of actual design presentation for a typical 5,000,000-gal. tank include complete stress analysis with diagrams showing application of forces, heat flow, and construction details.—The Preload Co., Inc., 211 East 37th St., New York, N. Y.

Mobile mixer and portable batcher

Drive-A-Mix, self-propelled mobile mixer and portable batcher is powered by a 26.8-hp engine. A tricycle steering design permits tight turns possible only

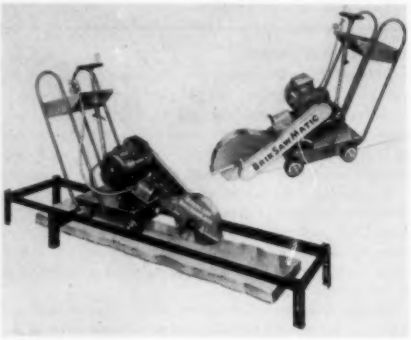


with manpower, concrete buggies, and wheelbarrows heretofore, says manufacturer.

The mixer carries its own water supply in 104-gal. capacity water tanks. Normally, five loads can be mixed without refilling. The mixer is 7 ft 8 in. high, 74 in. wide, and 11 ft 6 in. long. Gross weight is 3460 lb, and road speed is 18 mph.—Good-All Electric Mfg. Co., 120 First St., Ogallala, Neb.

Concrete cutting equipment

The Ready-Con Cart offers a means of converting masonry saws to light duty concrete saws or track saws. The manufacturer states that the cutting head can be mounted on bearing type pedestals on the cart; a crank on the dashboard of the cart mechanically feeds



the blade into the material being cut; and a water hose connection and valve are provided on the cart, together with a supply hose to connect the water valve to the cutting head.

The Ready-Con cutter now being produced is a complete, light duty electric concrete saw, ideal for use in plant maintenance work, according to the manufacturer.—Eveready BrikSaw Co., 1104 Union Ave., Kansas City, Mo.

Electrified cellular floor system withstands fire test

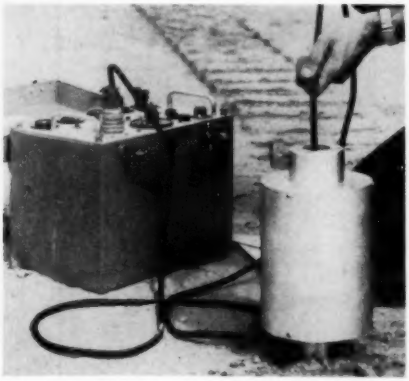
A completely electrified cellular floor system, built of E/R Cofar and standard Cofar units, has been subjected to an ASTM specified fire endurance test by technicians of Underwriters' Laboratories. The electrified area of the floor withstood fire requirements for 3 hr. Cofar is made of deep-corrugated galvanized steel sheets and the steel serves as a tight form for fresh concrete and becomes the main positive reinforcement when the concrete sets. The E/R Cofar units are steel troughs capped to form raceways through which wiring may be pulled to desk and office machine locations. Together these units formed the electrified cellular floor system tested. Fireproofing was established by ⅝ to 2 in. thickness of machine-sprayed perlite plaster under the floor beneath the electrification units. —Granco Steel Products Co., St. Louis, Mo.

Steel form catalog

A 16-page catalog gives specifications and sizes, and shows installations of steel forms for precast and prestressed concrete products. Describes in detail various multipurpose forms: adjustable square piling forms designed for mass production; 14-in. octagonal piling; sheet piling; bridge girders; double-T forms; I-beams; I-joists; flat slab (tongue-and-groove); tandem single-T; barrier (groin) form. Cable depresser shown can be attached to beam and girder forms; it positions strand or cable by hand wheel depressing action. Photographs show varied on-the-job applications of Watco forms. —Concrete Form Division, Plant City Welding & Tank Co., Plant City, Fla.

Portable radioactive moisture meter

A portable field instrument for rapidly measuring moisture content or density in a wide range of organic and inorganic materials enables a single operator to obtain accurate moisture or density determinations in 2 min., according to the manufacturer.



Basic units of the "d/M-gauge" consist of a dual purpose, 1½ x 15-in. probe containing radioactive source and detector system and a battery or a-c-operated scaler which counts and records impulses from the probe, registering total count in dots of light on five circular glow tubes. Measurements are obtained by inserting the desired probe (either a moisture probe containing a radioactive radium-beryllium source of fast neutrons, or a density probe containing a cesium-137 gamma ray source) into the material being tested and reading the scaler for a visual scatter count which varies with moisture or density variations.—Nuclear-Chicago Corp., 229 West Erie St., Chicago, Ill.

Membrane curing and coatings for floors

One-coat, penetrating finishes which combine the functions of curing agents, hardeners, and sealers and are applied before concrete has set, promise to reduce upkeep and dusting of concrete floors according to Harold V. Schmidt, Jr., of the Federal Varnish Division of Enterprise Paint Manufacturing Co., Chicago.

Speaking before the 43rd midyear meeting of Chemical Specialties Manufacturers' Association, he said that these membrane curing finishes act as water barriers to reduce the possibility of moisture being trapped between the floor slab and carpeting or other cover. Because the penetrating finishes minimize breakdown of the concrete surface, they also provide better adhesion between the concrete and mastic used when tile is to be applied over the concrete. He further added, however, that the new sealing coats are in no way designed as appearance items and for this reason the use of oleoresinous sealers and paints will continue. Other new developments in compounds for use on concrete floors, Schmidt said, include pigmented coatings which appear to have outstanding abrasion and chemical resistance properties. New resin emulsion coatings have also been developed for use on terrazzo floors where they contribute both lustre and safe footing conditions.—Chemical Specialties Manufacturers Association, 50 East 41st St., New York, N. Y.

Concrete dissolvent for cleaning tools

According to its manufacturer, Disolvex is a chemically compounded concentrate of organic and inorganic synthetics which works by catalytic action and commences to dissolve concrete immediately on contact. Suggested for cleaning tools, solution must surround the object to be cleaned at all times. The manufacturer further states that it has no effect on the aggregates; they are merely released and fall to the bottom of the receptacle. Manufacturer's tests show the usual cleaning time is from 4 hr for concrete up to ¼ in. thick and up to 6 months old, to 24 hr for older concrete. Producer notes that Disolvex is not dangerous to use, has no effect on any metals except aluminum, zinc, or magnesium, and does not affect rubber, plastics, or wood.—Industrial Synthetics Corp., 2000 West Walnut St., Chicago 12, Ill.

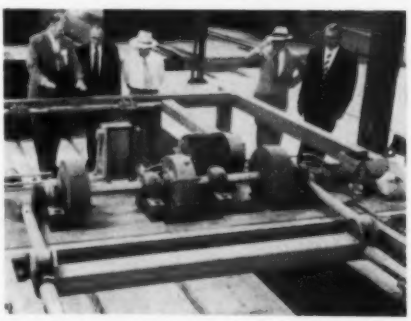
Expansion joint material

A recently introduced rubber expansion joint and caulking compound is said to elongate 325 percent through the normal temperature range of 70-80 F and maintain a tight seal of joints by squeezing and stretching with any movement. "Hornflex" is a two-component product mixed prior to use and applied either by caulking gun or knife. Manufacturer states that it sets to touch in 12 hr and attains maximum strength in 7 days, or less with damp curing.—A. C. Horn Co., Inc., Long Island City, N. Y.

Lightweight gas concrete

Durox gas concrete, developed by a Swedish firm, is now being used in the United States for manufacture of wall and roof elements as well as block and slabs. This gas concrete weighs 25 to 45 lb per cu ft, and according to manufacturer has a high water resistance, is frost resistant, and offers heat and sound insulation. Manufacturer reports that it can be sawed, nailed, bored, or hewn with ordinary carpenter's tools, or cemented together with mortar.

Durox is manufactured of lime and cement in combination with certain siliceous materials. Ingredients



are pulverized and combined with water; a fine aluminum powder is added as activating agent to set off chemical reactions producing as much as a four-fold volume increase. After initial set, material is cut by machines (see picture) which use steel wire as cutting blades; units are then steam cured.—U. S. Durox Corp. of Colorado, Englewood, Colo.

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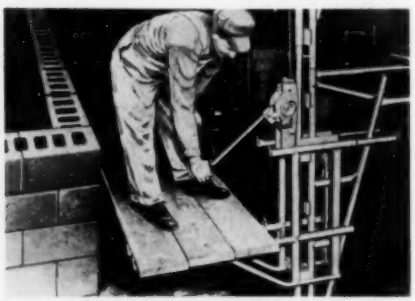
1958 truck mixers

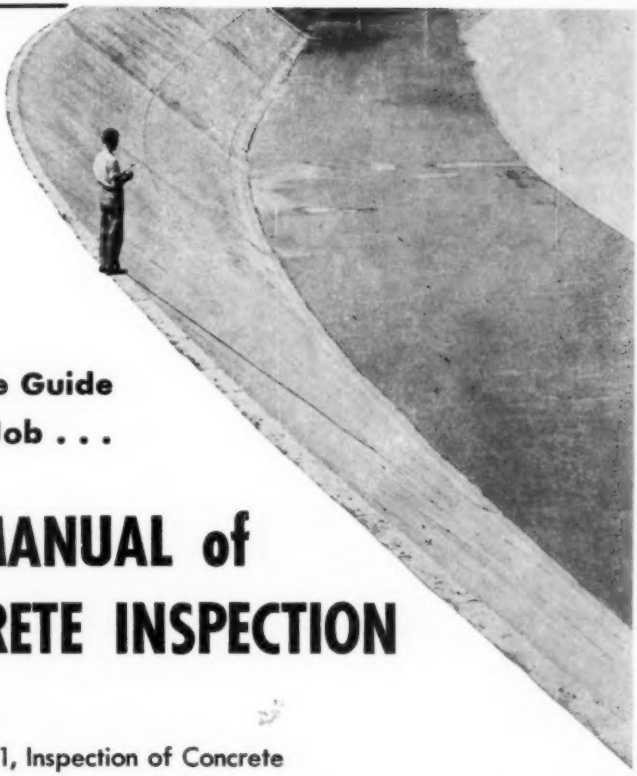
High-speed cycle, along with numerous advancements in design and operational simplicity, keynotes the 1958 Rex Adjusta-Wate Moto-Mixers, now in volume production. Single-lever operation with a new design instrument panel having all controls clustered close before the operator increases efficiency of the operator, according to the manufacturer. Hopper opening is larger and more steeply sloped; drum revolution speed has been increased; new fast shrink blade located in the drum head keeps mixed material at the front of the drum. The manufacturer advises that the quality of concrete received careful attention in designing the low-incline drum with greater free air mixing space and more effective water distribution.—Chain Belt Co., Milwaukee 1, Wis.

Winch-elevated platform for mason

Ezelift, a winch-elevated mason and material platform, or bracket, permits elevating of the work platform through four 6⅓-ft panel heights without repositioning the hanger, enabling the mason and his material to be at the proper height for each course of brick or



block. Manufacturer describes the bracket as a rugged, welded lightweight assembly providing a mason platform support adjustable from 20 to 30 in., and a similar material platform support with the same adjustment, elevated 20 in. above the mason's platform. The device is raised or lowered by a manually operated 4000-lb capacity winch with 24 usable ft of steel cable, on a drum with a safety tested ratchet.—Universal Manufacturing Corp., Zelienople, Pa.



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